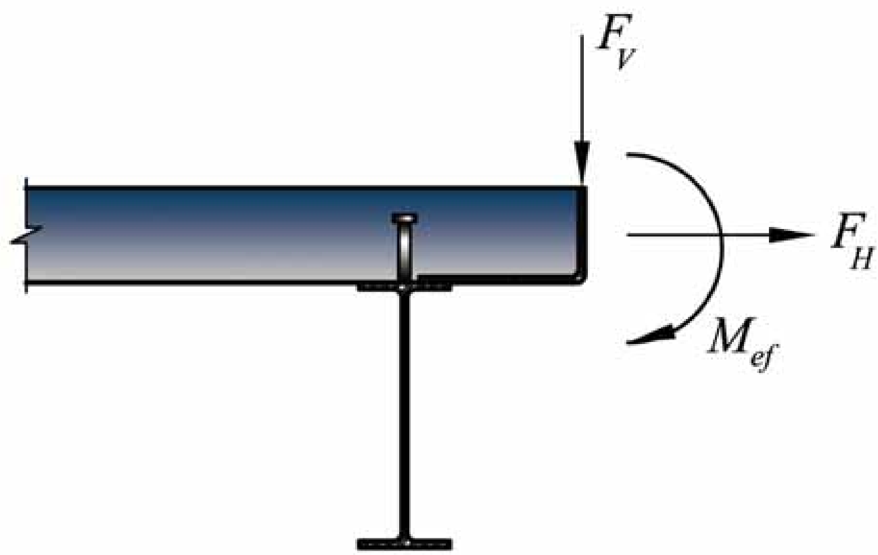
22

Steel Design Guide

Façade Attachments
to Steel-Framed Buildings



22

Steel Design Guide

Façade Attachments
to Steel-Framed Buildings

JAMES C. PARKER, P.E.
Simpson Gumpertz & Heger Inc.
Waltham, Massachusetts

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

Printed in the United States of America

ACKNOWLEDGMENTS

The author wishes to acknowledge the support provided to the author by Simpson Gumpertz & Heger Inc. during the development of this Design Guide. The information presented herein was derived from the collective knowledge of the firm, as well as from the noted references, and produced with the help of several of its dedicated and enthusiastic staff; namely: Alec Zimmer, Amy Schreiber, David Martin, Dirk Kestner, Kevin Guillotte, Michelle Meyer, and Matthew Johnson. The author thanks them for their collaboration and contributions.

The author also wishes to thank AISC and the following people for their assistance and review of this design guide. The Guide greatly benefited from their insight and suggestions.

Abbas Aminmansour
William A. Andrews
Paul M. Brosnahan
Charles J. Carter
Harry A. Cole
Theodore L. Droessler
Don Engler
Roger E. Ferch
Walter Heckel
Christopher M. Hewitt
W. Steven Hofmeister
William D. Liddy
William R. Lindley
H. Scott Metzger
Davis G. Parsons II
Victor Shneur
William N. Scott
Thomas S. Tarpy
Raymond H.R. Tide
Michael A. West

TABLE OF CONTENTS

Chapter 1
Introduction

Perhaps the most complicated details in a building occur where the façade and structural frame meet. The details of this interface have a significant impact on the cost of the project and performance of the façade. Performance issues that affect the façade attachment details include proper support of the façade elements, structural anchorage to the frame, relative movements, fire safing, waterproofing, thermal and moisture migration, air infiltration, and sound transmission. The design team must coordinate responsibilities among the architect, building frame engineer, façade engineer, general contractor, steel fabricator, steel erector, and façade subcontractor(s). This AISC Design Guide on façade attachments provides explanations of façade system fundamentals, highlights building performance issues that influence attachment design, and includes practical attachment design examples.

1.1 OBJECTIVE AND SCOPE

The objective of this Design Guide is to assist the practicing engineer in achieving economical slab edge details for steel frames that are structurally sound, durable, and accommodating of the performance requirements of the particular façade system. The focus is on façades—the non-load-bearing building enclosures attached to, and supported by, the building structure. This Design Guide presents concepts and fundamentals pertinent to façades in general, as well as specific information about supporting and anchoring some of the more common façade systems. Although primarily intended to assist the structural engineer responsible for design of the steel frame, this Design Guide is also a reference for the architect and the engineer responsible for the design of the façade elements.

When referring to the structural engineer responsible for the design of the steel frame, this Guide uses the term *structural engineer of record* (SER) as it is used in the AISC *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005). When referring to the engineer responsible for the structural design of the façade elements and/or their attachments, this Design Guide uses the term *specialty structural engineer* (SSE) in a manner consistent with that used by the Council of American Structural Engineers (CASE).

General concepts and principals of this Design Guide include façade performance fundamentals, attachment design criteria, roles and responsibilities, and fabrication and erection tolerances. Specific steel framing issues include slab-edge details and spandrel-beam design issues.

Specific façade systems include masonry cavity wall systems with concrete-block or steel-stud back-up, precast-concrete wall panels, aluminum curtain walls with glass and/or metal panels, glass-fiber-reinforced concrete (GFRC) and other lightweight panels, and exterior-insulation-and-finish-system (EIFS) panels.

No one text can present all of the creative and effective strategies and details that designers can and will develop, and this Design Guide does not represent an attempt to do this—nor is it an attempt to present preferred details. Preference depends on the specific conditions for a given project, regional norms, and individual designers, fabricators, and erectors. Rather, the concepts and performance characteristics that will lead to successful support of façades are described. By way of illustrative sample details and example problems, readers will see how to implement these concepts and achieve proper performance. This, along with a basic understanding of fundamental principles, will help the practicing engineer to develop and apply sound strategies for support and attachment of a façade on a particular project, addressing any number of project-specific conditions.

This Design Guide focuses on attachment strategies and their effect on the design, fabrication, and erection of steel frames. Although the general background is presented on various façade systems and principles for their proper support, this Design Guide does not focus on the design of the façade components, their intra-connections, or anchors integral to the façade structure, such as embedded inserts into concrete panels or flex anchors of GFRC panels.

1.2 FUNDAMENTALS OF FAÇADE PERFORMANCE

1.2.1 The Façade and the Building Envelope

The building envelope encloses the building, controlling the transmission of air, water, heat, sound, and light, both into and out of the building. The exterior walls, roofs, windows, doors, foundation walls, and foundation slabs, and the interfaces of these parts, comprise the building envelope. The exterior wall is but one of the envelope components and the façade is just one component of the exterior wall. However, when this Design Guide refers to façades and façade attachments, it is meant to encompass all those components of the exterior wall supported by and anchored to the building, either directly or indirectly through other wall components.

The functional components of the exterior wall include:

- The cladding—what can be seen from the outside;
- The structure of the wall, which may be integral with the cladding or an independent backup wall or frame behind the cladding;
- Water barriers, air barriers, and vapor retarders;
- Joints between components;
- Insulation; and
- Interior finishes.

The extent to which each of these functional components is an independent physical component depends on the wall system. Many times, one physical component performs several functions in the wall. For example, a precast-concrete panel can function as the cladding, structure, water barrier, and vapor retarder of the system.

Together, the physical components comprise the exterior wall assembly, which:

- Accommodates structural loads and deformations from its self weight in addition to, wind, seismic, and thermal loads applied to it;
- Minimizes water penetration and air flow;
- Controls heat gain or loss and water vapor movement into or out of the building; and,
- Accommodates differential movement between the wall components, and between the wall assembly and the primary building structure.

The strategy and methods that the designers choose to use to support and attach the façade to the structure must not compromise the ability of the assembly to perform as intended. The methods must also apply the loads to the primary building structure in a manner that is consistent with the design of the primary building structure.

1.2.2 Concepts for Control of Water Infiltration

Controlling water infiltration is usually the most important factor for the durability and performance of the wall system. The water barrier is the most critical element for stopping water infiltration, but insulation, air barriers, and vapor retarders also play a role in controlling water that may condense in the wall. It is helpful to understand the concepts for stopping water when designing façade attachments to ensure that the support strategy does not compromise the water barrier and control concepts.

The location of the water barrier depends on the concept(s) used in the wall system to control infiltration. Four concepts will be discussed: barrier systems, internal drainage planes, cavity walls, and pressure-equalized rain screens.

Barrier Systems

Barrier systems rely on the wall material or cladding material to prevent infiltration without the benefit of drainage or internal water barriers (waterproofing membranes). Historic masonry walls are classic examples of this approach. These walls rely on their massive thickness for sufficient moisture control (sufficient for the times!). Modern examples can be found in precast-concrete panels, GFRC panels, and EIFS panels, although varieties of all three systems can be found with drainage back-ups. Actually, any cladding system that relies solely on the exterior surface to prevent water from entering the building is considered a barrier-wall concept. Barrier systems rely heavily on the performance of the joints between panels and components where the barrier surface is interrupted. Designers should give close attention to joint movement and preventing stresses that could damage the barrier (such as cracking in precast-concrete or GFRC panels). Barrier systems place high reliance on near-perfection of the wall barrier. While common, some designers consider them inappropriate for installations where reliable waterproofing is required.

Internal-Drainage-Plane Systems

This concept provides a water barrier behind the cladding with a narrow drainage plane between them. Traditional stucco walls employ this concept. A water barrier, often asphalt-saturated building paper, is applied over the exterior face of the back-up wall. The plaster and lath is applied in such a way that a drainage plane is formed between the stucco and building paper.

Internal drainage planes can be used to enhance the barrier concept. For example, some modern EIFS panels now incorporate a drainage plane. Many metal and aluminum curtain walls are also examples of barrier systems that are enhanced by provisions to drain water that penetrates the exterior surface. Flashings are designed to divert water at the drainage plane back out through weeps.

Cavity-Wall Systems

Similar in concept to internal-drainage-plane systems, this concept employs a wide air space between the back of the cladding and the water barrier. The cladding need only control the volume of water in the cavity as the water barrier and flashings are designed to divert water in the cavity back out. This concept is used with brick and stone veneers and metal panel systems.

Pressure-Equalized Rain Screens

The term "rain screens" is sometimes used to describe systems that are actually internal-drainage-plane or cavity-wall systems that use cladding with open joints. The cladding is considered just a "screen" to minimize moisture, and the protection is actually the water barrier behind the cladding. Pressure equalization refers to designs where the cavity is compartmentalized and vented in such a way that the internal cavity pressure instantaneously is similar to the exterior pressure, preventing significant amounts of rain from entering the cavity despite the open joints. This has the obvious benefit of not needing to seal the joints or maintain them. However, designing a wall to achieve pressure equalization under all conditions is uncertain and complex enough that most designers employ water barriers and drainage measures behind the screen, and do not rely solely on pressure equalization. Thus the term "rain-screen" may be used even if the pressure-equalization concept is not employed.

Flashings are part of the water barrier and required at all penetrations of the barrier, such as windows and doors; at interruptions, such as shelf angles; and at all terminations, such as at interfaces with the roof, foundations, or other wall systems.

Problems in the water barrier and/or flashings associated with the support and anchorage of cladding include:

- Anchors or support clips interrupting the flashing, or interrupting the water barrier without flashing or proper repair of the barrier;

- Anchors or supports causing conditions of poor drainage (barrier and flashing surfaces should be sloped to drain, and contouring the barrier or flashing around supports can inadvertently lead to areas that don't drain);

- Differential movement between the wall system components or the wall and the structure such that the barrier or flashings, or their seams, are torn;

- Damage to the barrier or flashing during erection and installation; and,

- Constructability issues regarding the sequencing and coordination of trades responsible for anchorage and waterproofing considerations.

1.2.3 Vapor Retarders and Air-Barrier Systems

Vapor retarders and air-barrier systems also play a role in the proper performance of the wall as part of the building envelope. In the past, designers relied on the application of vapor "barriers" (actually only retarders) to mitigate condensation of moist air in the wall system. Vapor retarders reduce the movement of moisture through the wall by having a low permeability. Today, many designers realize that most moisture is transferred through air movement, and that the air-barrier system has the most effect on moisture migration through the wall. The continuity of the air-barrier system is critical.

Whereas a few flaws in the vapor retarder have little effect on the amount of water that passes through the material, a few defects in an air barrier with a pressure differential across it causes air to flow through the defects, carrying large amounts of moisture through the system. Many films and common materials can be air barriers; yet achieving a complete air-barrier system requires attention to the details of joints, penetrations, and terminations.

It is beyond the scope of this Design Guide to provide recommendations for choosing and placing air-barrier systems and vapor retarders. However, it is important that the support and anchorage design consider the consequences of breaches in the air-barrier system. This is especially true for humidified spaces and other moisture-rich conditions, such as natatoriums.

1.2.4 Insulation and Thermal Performance

The thermal performance of the building envelope is ever gaining in importance and attention. Building codes for new construction are increasing the requirements placed on building enclosures. Structural attachments that cause thermal bridges not only compromise the thermal performance but also can lead to moisture problems due to condensation. The design of supports and anchorage should avoid thermal "short circuits," when practical. When this is not possible, designers of the wall system may need to give the conditions special consideration, employing thermal and moisture migration modeling of the system to understand the effect on energy performance, and the potential for condensation on cold elements.

1.2.5 Sealant Joints

Joints are necessary in all façade systems. They are usually necessary to fabricate and install the façade accounting for tolerances and clearances; accommodate thermal and moisture movements; accommodate façade deformations from gravity, wind, and seismic forces; and accommodate differential movement between the primary building frame and the façade. The layout of the joints is not only a key element in the look of the façade, but also a key parameter for selecting the support strategy.

Although the most reliable wall systems do not rely solely on sealant joints as part of the envelope water or air barrier, sealant joints are still an important part of system durability. In less-redundant systems, the sealant joints may be critical to the façade system performance.

Sealant joints perform two functions simultaneously, allowing for movement and stopping water and air entry. The performance and durability of sealant joints depends on the bond of the sealant to the substrate, the tooled shape of the

sealant, and the sealant movement capability. The movement capability is the amount the sealant can compress or stretch without failing, and is expressed as an absolute percentage from the original size of the joint. High-performance sealants can accommodate movements of 25 to 50 percent of their original size. The movement capability must account for two sources of movement: movements of the facade itself due to imposed loads and differential movements between the faced elements and the supporting steel frame due to thermal and moisture changes. The total design joint width should also include allowances for construction tolerances.

The following formula can be used to calculate the width of a sealant joint:

$$J' = J + \delta_{ps} + T \geq ½ \text{ in.}$$

where

J = $(\alpha \, \Delta_T \, L + k_e \, L + \delta_{sil})/M$

then

$$J' = (\alpha \, \Delta_T \, L + k_e \, L + \delta_{sil})/M + \delta_{ps} + T \geq ½ \text{ in.}$$

where

J' = design width of the gap prior to sealant installation, in.

J = minimum width of the sealant joint, in.

α = coefficient of thermal expansion of the façade material, in./in./°F

Δ_T = design temperature change, °F

L = length of material between joints, in.

k_e = coefficient of moisture expansion, in./in. (applicable for brick)

δ_{sil} = design movements (such as structural deflections due to superimposed loads) that occur after the joint is sealed, in.

M = movement capacity of the sealant material, expressed in percent

δ_{ps} = the relative deflection of the structure with respect to the brick below the shelf angle that occurs after shelf angle is set but before the joint is sealed, in.

T = required construction tolerance for the joint, in.

The minimum joint width is ½ in. This minimum dimension is needed to achieve most sealant manufacturer recommendations for minimum bond area, and for depth-to-width ratio in the sealant. A sealant joint profile is illustrated in Figure 1-1.

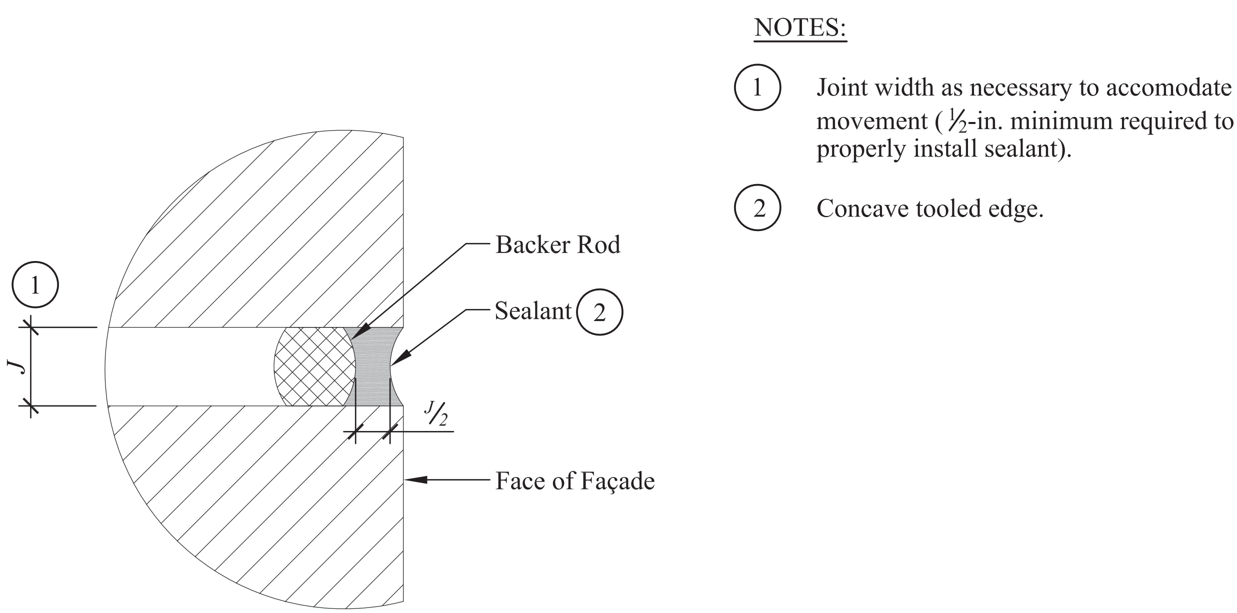


Fig. 1-1. Sealant joint profile.

Chapter 2
General Design Criteria for Attachment of Façades

General design criteria for the façade and exterior wall system on a building can be divided into three categories: structural integrity, provisions for movement, and envelope performance. The attachment strategy for the façade plays a role in each of these categories. This chapter presents design criteria for the attachments to the steel frame, focusing on general criteria common to most façade systems.

When the design team is developing an attachment strategy, the primary design criteria for the attachments include:

- Structural integrity (strength, ductility, and redundancy);
- Accommodating movements of the façade and frame;
- Durability;
- Accounting for tolerances and clearances; and
- Constructability and economy.

Often, ideas for details to accommodate one criterion will be in conflict with other criteria, and a satisfactory balance of the competing needs is necessary to arrive at a successful attachment design.

Building codes, such as the 2006 *International Building Code* (ICC, 2006), provide criteria for the determination of the dead, live, wind, and seismic loads, often by reference to ASCE 7 *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2005). The material design standards from AISC, ACI, and others are also referenced, and provide the corresponding design criteria for strength to resist the loads. However, the building codes generally do not provide guidance addressing the other design criteria listed above. ASCE 7 has provisions for estimating relative displacements due to seismic loads, as well as ensuring adequate strength and deformation capability for component connections when subjected to seismic drifts. However, no direction is provided on serviceability limits for movement due to wind, temperature, and moisture, all of which affect the performance of the façade as a building envelope.

References and standards are provided by the various façade-system-related trade associations, including design criteria. Although these resources generally provide sound advice on the design of the façade itself, they generally give casual attention to the design of the façade attachments to the primary building structure, and particularly little attention to the effects the attachments have on the primary building structure, such as eccentricity or impact on fabrication or cost. Most details in the trade literature show the primary building structure in only a schematic way. This Design Guide, having the advantage of being specific to steel frames, addresses design criteria for façade attachments and their impact on the primary building structures.

2.1 STRUCTURAL INTEGRITY

Attachments achieve structural integrity when they have sufficient strength, ductility, and redundancy. The connections must have adequate strength to safely resist applied forces, as well as sufficient inelastic deformation capacity. Ideally, the failure of any one connection of the façade element will not lead to total loss of attachment to the building.

Unfortunately, achieving a balance of all three—strength, ductility, and redundancy—can be difficult. For example, in the attachment of panelized façade systems it is desirable to support the panel for gravity loads in only two locations (this is explained in later chapters), which potentially reduces redundancy. Also, the connection of the attachment to the panel may be made with anchors and/or inserts that have low deformation capacity, and alternative choices may affect other design criteria, such as economy or envelope performance. When designers are faced with conditions that compromise ductility or redundancy, a much greater confidence is required that sufficient strength is provided, perhaps by using a larger factor of safety (larger load factors) and more stringent quality assurance (inspection and testing) requirements.

2.1.1 Gravity Loads

The dead load of the façade is usually the dominant gravity load for attachments. Most façades carry no live loads. Façades that have horizontal projections, however, may have snow, rain, or ice loads. In addition, window-washing activities may impose live loads onto façade systems.

ASCE 7 provides guidance on minimum dead loads of materials and material densities, but gives little specific attention to façade assemblies. The AISC *Manual* (AISC, 2005c) also provides estimates of material weights, including for some wall assemblies. Both of these resources are a good place to start, as is Table 2-1 in this Design Guide.

Table 2-1. Dead Loads for Façade Components and Systems

Component				Load	Common Design Assumption for System
Coverings		Bituthene Membrane		0.4 psf	5 psf
		Extruded Polystyrene, 2 in.		0.3 psf	
		Gypsum Sheathing, ⅝ in.		2.5 psf	
Brick Veneer, 4-in. Wythe, (40 psf)	6-in. or 8-in. Metal Stud Back-up	18 Gage	24-in. o.c.	1.1 psf	45–50 psf[1]
			16-in. o.c	1.7 psf	
	Concrete Block Back-up (130-pcf Density)	6-in. Wythe	No Grout	30 psf	75–85 psf[2]
			48-in. o.c. Grout Spacing	36 psf	
			40-in. o.c. Grout Spacing	37 psf	
			32-in. o.c. Grout Spacing	38 psf	
			24-in. o.c. Grout Spacing	41 psf	
			16-in. o.c. Grout Spacing	46 psf	
			Full Grout	62 psf	
		8-in. Wythe	No GProut	39 psf	85–100 psf[2]
			48-in. o.c. Grout Spacing	47 psf	
			40-in. o.c. Grout Spacing	48 psf	
			32-in. o.c. Grout Spacing	50 psf	
			24-in. o.c. Grout Spacing	54 psf	
			16-in. o.c. Grout Spacing	61 psf	
			Full Grout	83 psf	
		12-in. Wythe	No Grout	54 psf	105–130 psf[2]
			48-in. o.c. Grout Spacing	66 psf	
			40-in. o.c. Grout Spacing	69 psf	
			32-in. o.c. Grout Spacing	72 psf	
			24-in. o.c. Grout Spacing	78 psf	
			16-in. o.c. Grout Spacing	90 psf	
			Full Grout	127 psf	
GFRC Panels		120-pcf Density	½-in. Backing	5 psf	9–25 psf[3]
			⅝-in. Backing	6.3 psf	
			⅜-in. Exposed Aggregate Face	3.8 psf	
		140-pcf Density	½-in. Backing	5.8 psf	
			⅝-in. Backing	7.3 psf	
			⅜-in. Exposed Aggregate Face	4.4 psf	
Precast Concrete Panels (150-pcf density)		4-in. Panel		50 psf	55 psf[4]
		6-in. Panel		75 psf	80 psf[4]
		8-in. Panel		100 psf	105 psf[4]
Aluminum Curtain Walls		Curtain-wall Framing		3 plf	10 psf
		Storefront Framing		1.8 plf	
		Insulating glass, 1-in. Total Thickness[6]		6.8 psf	
Metal Insulated Panels		1¾-in. Panel		2 psf	10–15 psf[5]
		4-in. Panel		4 psf	
EIFS		Self Weight		1 psf	10 psf[5]

Notes:
1. Values include waterproofing, insulation, gypsum sheathing, and hardware.
2. Values are for solid walls; deduct for window and other openings.
3. Values include insulation, gypsum sheathing, metal studs, and hardware.
4. Values include insulation and hardware.
5. Values include gypsum sheathing, metal studs, and hardware.
6. 1-in. total thickness includes ¼-in. glass, ½-in. air space, and ¼-in. glass.

If the designer knows the components of the façade system in detail, the designer can derive a close estimate of the façade dead load. However, at the time the SER is designing the frame and developing attachment strategies for the façade, the façade is usually not designed and the SER must estimate the weight of the system. A modest amount of conservatism is warranted. The SER must judge the degree of uncertainty about the façade system or systems, along with the consequences of under- or over-estimating, to determine just how conservative to be. Table 2-1 provides example weights of common exterior wall and façade assemblies. The SER should consider including the basis for the façade loads used when developing the contract documents, especially when the façade system has not been selected.

Loads from window-washing activities are a function of the window-washing methods and equipment, which are governed by OSHA and state and local regulations. Although these forces are important for the design of the façade elements, it is rare that they govern the design of the façade attachment to the frame. Designers should be cognizant of unusual conditions where window washing loads may be large with respect to other loads and affect the design.

The strategy that the designer employs to resolve the eccentricity between the center of gravity (CG) of the façade and the line of support from the frame has a major effect on the design of both the façade and the supporting structure. It is often easiest to design attachments so that there is a theoretical hinge (inflection point) at some point between the façade and the structure to avoid the interaction of forces being dependent on the relative rigidities of the frame and façade.

Figure 2-1 illustrates the effect of the location assumed in the design of the façade attachments. The total eccentricity between the CG of the façade and line of support is the sum of the distances between the hinge and the line of support, e_s, and between the hinge and the CG of the façade, e_f. The effects of eccentricity, e_s, are resolved in the support structure, and the effects of eccentricity, e_f, are resolved in the façade structure with overturning stability provided by horizontal reactions to the support structure. Although M_{es} is shown as a moment on the spandrel beam, this moment often can be resolved with flexure in the slab.

Design criteria should establish how much of the total eccentricity will be taken by the façade, and this criteria should be included on the contract documents. The SER may wish to consider the economical effect of designing the support structure for the upper and lower bounds of where the hinge may be, leaving more latitude to the façade designer to design the façade and attachments.

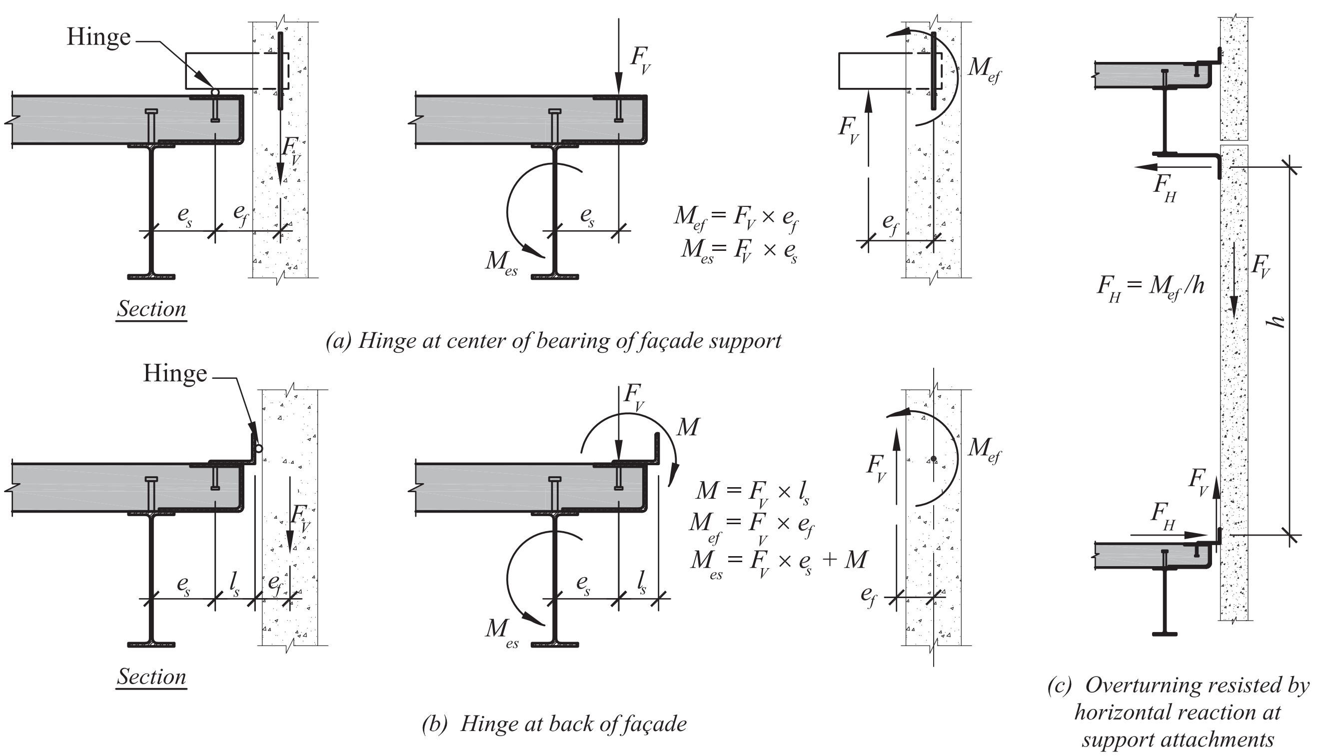


Fig. 2-1. Effects of assumed hinge location and eccentricity on structure and façade.

2.1.2 Wind Loads

Wind loads are established by prescriptive procedures or wind-tunnel testing procedures specified by the Applicable Building Code, which often refers to ASCE 7 or has similar wind provisions. Designers should bear in mind the following points with respect to wind loads on attachments:

- Design wind pressures on cladding are a function of the tributary area to the component.

- Design wind pressures are considerably higher near corners. Consider if the attachment strategy for the typical areas is scalable to work at the corners, or if an alternative strategy is needed at the corners. It is desirable to use a strategy that is scalable, using larger or more fasteners instead of new concepts in the detail.

- Design wind pressures are both positive and negative. A load combination with minimum dead load and negative pressures may change the direction of load and control the design of attachments.

- Structures with irregular shape or unusual response characteristics warrant special attention in the design for wind loading. Wind-tunnel tests or design based on literature specific to those unusual wind-load effects may be required. Wind-tunnel tests can be used to justify lower cladding loads, but can also reveal higher pressures that are more accurate for the particular structure. For large projects, especially for high-rise construction, wind-tunnel tests can result in significant savings for the façade and its attachments.

2.1.3 Seismic Loads

As with wind loads, seismic loads are established by the Applicable Building Code, which often refers to ASCE 7 or has similar provisions. Designers should bear in mind the following points with respect to seismic loads on attachments:

- The wall-component seismic forces increase with the height of the component on the building.

- The connecting members should have sufficient ductility and rotation capacity to preclude fracture of concrete and masonry, or brittle failures near fasteners or welds.

- Bolts, welds, and other fasteners in the connecting system are usually designed for forces that are amplified relative to the wall element. This is to ensure that the failure does not occur in the fastener, which promotes ductility. ASCE 7 requires that fasteners be designed for forces that are 3.1 times larger than the wall-element forces.

- Current seismic design requirements have special provisions for the anchorage of concrete and masonry walls that are laterally supported by flexible diaphragms. These provisions increase the anchorage design forces.

2.1.4 Loads from Restraint of Movement

When the façade attachment is such that the attachments restrain panel movement from thermal expansion and contraction, moisture-related volume change, or other volume changes, attachments will be subjected to forces from these restrained movements. It is usually desirable to avoid this restraint with details that allow movement. Determining forces from restrained volume change is difficult and inexact, as the forces are very dependent on small variations of attachment stiffness and such factors as façade component cracking and creep. Additionally, forces from restrained movements from thermal and moisture effects can be cyclical and necessitate fatigue considerations.

Designers should not overlook the potential for forces from inadvertent restraint due to friction or binding at sliding connections.

2.2 ACCOMMODATING RELATIVE MOVEMENT

Failure to provide for movement relative to both adjacent façade components and the supporting frame is the root cause of many instances of façade performance failure. Sources of movement include:

- Gravity loads, which cause spandrel deflections and rotations, column shortening, attachment bracket deflections, and/or façade deformations;

- Wind and seismic loads, which cause inter-story drift and/or displacements at building expansion joints;

- Temperature changes, which cause thermal expansion and contraction of the façade and building frame, as well as bowing from temperature differential through the thickness of the façade elements; and,

- Moisture changes and other volume changes.

The performance of the façade envelope in accommodating these movements relies on the design of the joints and the points and methods of attachment.

Designers must determine the design movement at the joints to select the joint configuration and material. The design movement of the joint depends not only on the sources of movement but also on the layout of the joints. Smaller joint spacing means smaller movements due to temperature and moisture at each joint. Strategic layout of joints

relative to the façade and frame geometry can minimize the effects of the movement at joints due to frame drift. The layout of the joints in turn determines the strategy for attachment to the building structure because, for most façade systems, each panel requires independent support and attachment.

Designers have traditionally used rules-of-thumb for flexural stiffness criteria of spandrel beams. The criteria may limit live-load deflections to between $L/360$ and $L/600$, depending on the façade material, to protect the façade material from cracking. However, controlling joint movement may be a more critical concern.

For example, a ¾-in. horizontal sealant joint that separates panels supported on a lower floor from panels supported on the floor above may have an allowable repetitive movement of ¼ in. (allowable movement capacity of 33⅓ percent). If project conditions are such that the anticipated thermal and moisture movement is ⅛ in., this leaves ⅛ in. for structural movement. Assuming only live loads on the primary structure move the joint subsequent to filling it with sealant, the deflection limit for live loads is then ⅛ in. Because this is a serviceability check, judgment must be exercised and perhaps the designer decides that using 50 percent of the live load is appropriate. The live load deflection limit for full live load is then ¼ in. This equals $L/960$ and $L/1440$ on 20-ft and 30-ft spans, respectively.

Obviously, designers can achieve economies in the steel spandrel girder design if joint layout and sizes are such that they do not control the design. However, extra-wide sealant joints affect the look of the façade, are costly, and have construction and performance issues. The design team must balance these issues with the cost of stiffer framing. Figure 2-2 illustrates the effects of spandrel deflection on panel joints.

Façade attachments must accommodate the relative movement between floors due to the lateral drift of the frame as a result of wind and seismic forces. ASCE 7 requires that connections and panel joints allow for the story drift caused by seismic displacements, or a minimum of ½ in. The same provisions allow movement to be accommodated by slotted or oversized holes, bending of steel, or other equivalent sliding or ductile behavior. Although not explicitly cited in IBC or ASCE 7, similar allowances for drift should be provided for wind forces. Designers should avoid accommodating wind drift by bending of steel parts that result in yielding without evaluation of the potential for low-cycle fatigue.

Inter-story drifts that occur out-of-plane to the façade are accommodated by rotations at the attachments, or by flexure of the façade, as demonstrated in Figure 2-3. Wind-drift effects on attachments typically are small when the story drift is limited to $H/400$ or $H/500$. However, code-permitted seismic drifts may be as high as $H/40$, or 10 or more times

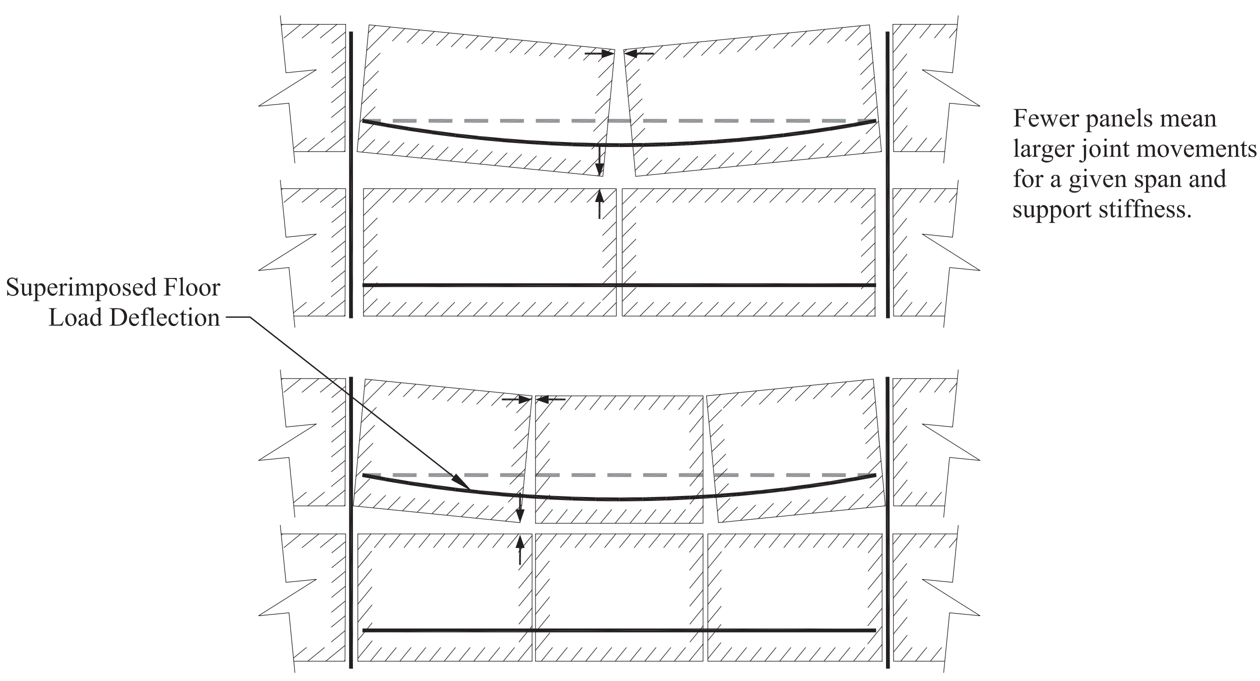


Fig. 2-2. Effects of spandrel deflection on panel joints.

these wind limits. Thus, it is important to design the attachments to accommodate seismic drifts that cause out-of-plane movement of the wall. Usually, the attachments are designed to allow rotation and/or resist moments induced from the lateral displacement and flexural force in the façade.

Inter-story drift causing movement in the plane of the façade is best accommodated by allowing for slip along a horizontal movement joint. Façade panels should have attachments that restrain them in-plane at only one floor. If panels require attachment to another floor for out-of-plane stability, these attachments should allow movement in the plane of the façade. Figure 2-4 illustrates panel layout and its effect on joint movement for in-plane drift.

Corners require special study for joints. Joints designed for in-plane and out-of-plane movements in the field of the wall may not be sufficient at the corner as in-plane movement becomes out-of-plane movement, and vice versa. Figure 2-5 illustrates movements at the corner of a building.

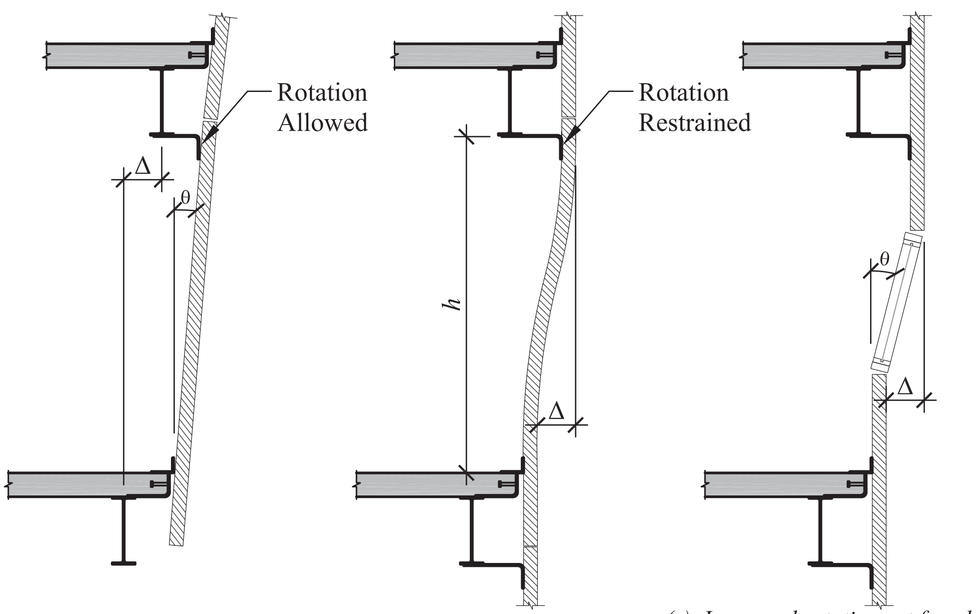
Sometimes façade elements are supported on the foundations and the façade elements are self-supporting vertically, using the steel frame only for out-of-plane support (for example, concrete wall panels). When the façade panels stack more than two tiers, accommodating the in-plane relative seismic drift between the frame and façade elements is difficult unless the frame is made unusually stiff.

The code-prescribed wind and seismic forces are for events with mean recurrence intervals of 50 and 475 years, respectively. Attachments must safely accommodate movements, and joints must be designed to prevent hazardous damage to the façades from these levels of forces. However, serviceability checks are normally made with lower-level forces. Designers should select appropriate performance objectives after consultation with the owner.

ASCE 7 suggests in its commentary on serviceability considerations that design using the drift at full code-specified wind loads is excessively conservative. ASCE 7 further suggests that the appropriate load combination for serviceability checks for short-term effects is:

$$D + 0.5L + 0.7W$$

Note that this combination has an annual probability of 5 percent of being exceeded, which translates to 72-percent and 92-percent probabilities of being exceeded in 25 years and 50 years, respectively. Designers and owners should consider these relatively high probabilities that the load case will be exceeded sometime in the life of a building and be sure the consequences of being exceeded are acceptable.



(a) Rotation at attachments (b) Flexure of façade element (c) Increased rotations at facade sub-element such as a window

Fig. 2-3. Effects of out-of-plane inter-story drift on façade panels.

Perhaps damage control and functioning sealant joints are acceptable for the 5-percent wind event if the structural aspects are designed for the load combinations corresponding to a 50-year mean recurrence interval.

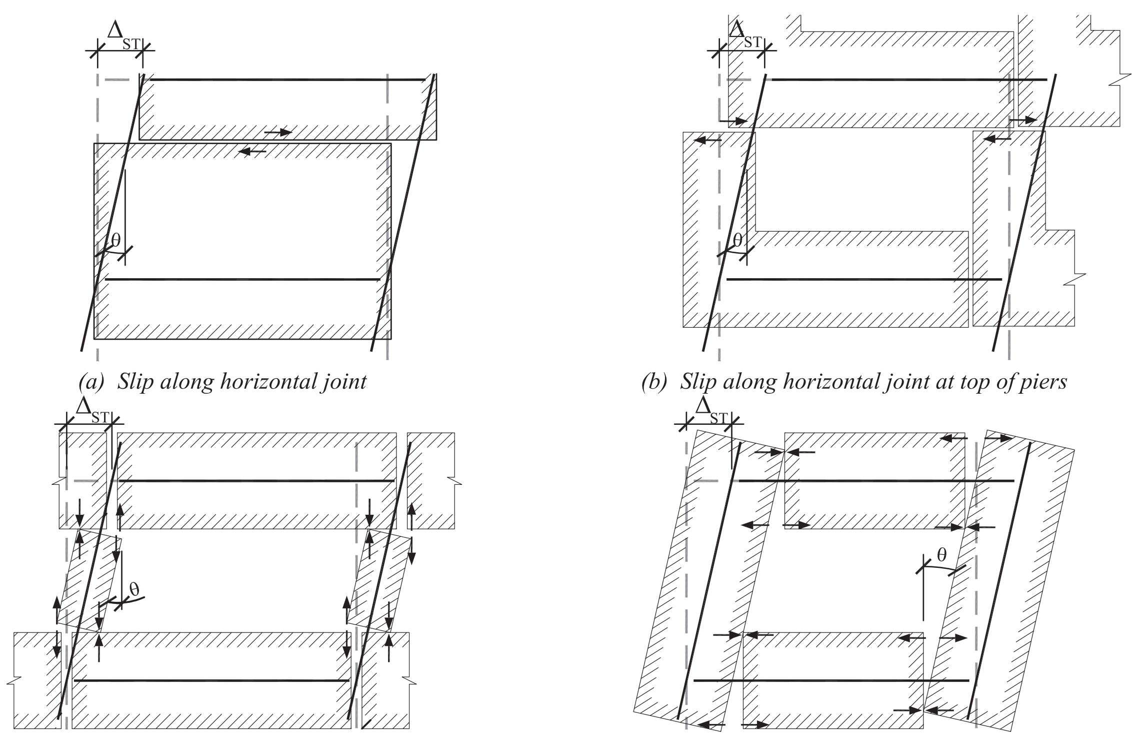
Griffis (1993) addresses serviceability limit states for wind loads on buildings, including the selection of appropriate drift limits. One point also germane to attachments of façades is that evaluating inter-story drift alone can be misleading. Total inter-story drift is made up of shear and flexural deformations of the frame. The shear deformations are usually the major source of potential damage to façade systems. Flexural deformations are less of a problem because they result in mostly rigid body motion of the façade elements within the frame panel. This is illustrated in Figure 2-6.

2.3 DURABILITY OF ATTACHMENTS

Façade attachments are usually difficult to inspect during the life of a building. To do so may require removal and/or replacement of façade parts, or of the back-up wall, to visually inspect the attachments and their anchors. Furthermore, shortcomings in the façade design and/or construction may lead to water leaks that expose parts within the wall that were otherwise intended to be dry. Since failure of an attachment could lead to falling hazards, due consideration should be given to durability as a design criterion.

The measures that should be taken to ensure the durability of attachments to the frame depend upon the water management philosophy of the façade system and the likelihood of the attachments and anchors being exposed to water. Attachments behind a waterproof membrane in a system that employs a drainage layer or cavity are significantly less likely to be exposed to water than attachments for a face-sealed panel system. Designers should also assess the potential for water to pool on the attachment or otherwise expose the attachment to water over a long period of time, even if the leakage is intermittent. Unless there is a high level of confidence that the attachments will remain dry, corrosion protection should be specified for the attachment parts. Thinner elements, such as light-gauge metal clips, require greater protection than heavier structural steel clips to achieve the same level of durability. If conditions warrant corrosion protection, attachments can be specified as hot-dip galvanized steel or stainless steel.

Hot-dip galvanized steel attachments are usually less expensive, and can be made from plates and shapes that are usually more readily available than similar parts in stainless



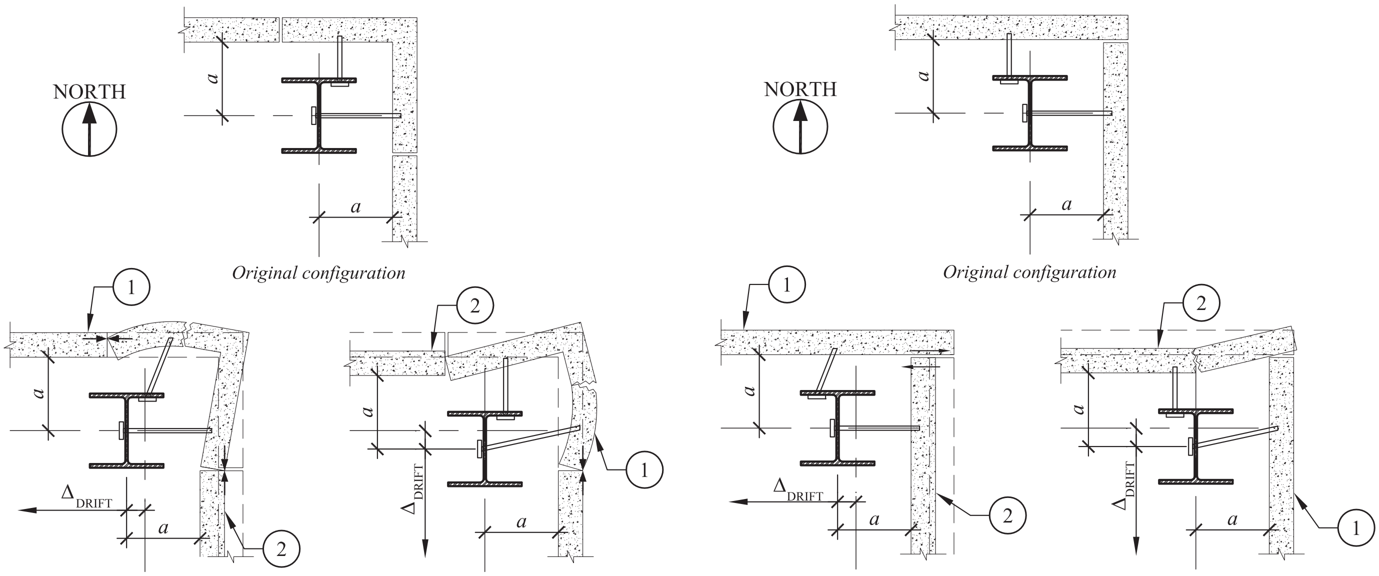
(a) Slip along horizontal joint

(b) Slip along horizontal joint at top of piers

(c) In-plane drift causing joint compression/tension

(d) In-plane drift causing joint compression/tension

Fig. 2-4. Effects of panel layout on joint movement for in-plate drift.



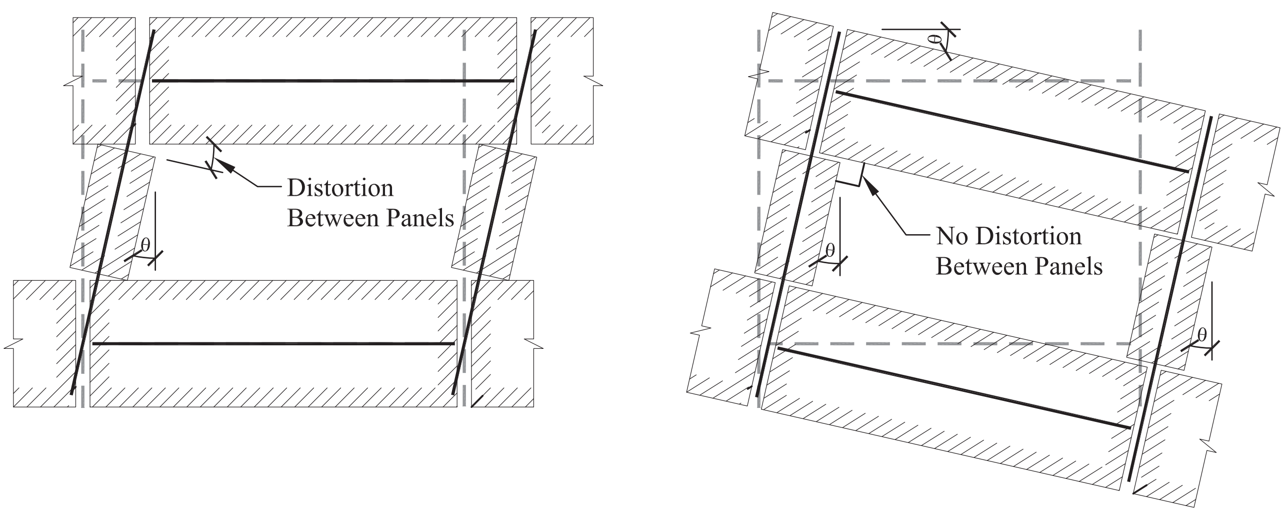
Distortion in East/West plane *Distortion in North/South plane* *Distortion in East/West plane* *Distortion in North/South plane*

NOTES:

1 In-plane movement of wall is with floor below.

2 Top of wall moves out-of-plane with floor above.

Fig. 2-5. Inter-story drift effects at corners.



(a) Frame shear deformation *(b) Frame flexural deformations*

Fig. 2-6. Inter-story drift effects on façade panels.

steel. Field welding of galvanized parts also involves more common welding procedures than those for stainless steel parts, provided the proper steps are taken to remove the galvanizing at the weld location and then repair with a zinc-rich coating afterward. Care must be taken, however, as corners, edges, or other incidental defects where the zinc coating may be lost can corrode, and the rust product may wash away and stain visible parts of the façade.

Galvanized steel is generally available with specified minimum yield strengths of 36 ksi and 50 ksi. Stainless steels are available in significantly higher strengths but at higher costs. Also note that the zinc coating of galvanized parts is a sacrificial coating with a defined service life that depends on the environment in which it is used.

2.4 ACCOUNTING FOR TOLERANCE AND CLEARANCES

One of the most difficult design objectives to accomplish is having sufficient provisions in the details for tolerances and clearances. Tolerances refer to the permissible amount of deviation from a specified criterion (dimension, shape, location, etc.). Clearances refer to space purposely provided between adjacent parts to allow for movement, accommodate tolerances, and provide access, if needed, for installation of attachment hardware.

For the design of façade attachments to steel frames, the sufficient adjustability and clearance must be provided to allow the façade to be erected within its tolerances relative to the theoretical plane of the façade. The magnitudes of façade tolerances are usually significantly smaller than the magnitudes of the tolerances applicable to the fabrication and erection of the steel frame. For example, the exterior line of the steel frame may be within its tolerances of 1:500 but not more than 1 in. outward and 2 in. inward for a building less than 20 stories, and the tolerance of the curtain wall may be plus or minus ½ in. Therefore, adjustment provided in attachments must account for the steel frame tolerance, and the differences between the tolerances for the steel frame and the façade, for the wall to be erected within its tolerance.

Chapter 4 provides a more detailed discussion of steel frame tolerances and building façade system tolerances.

2.5 CONSTRUCTABILITY AND ECONOMY

Presenting constructability and economy as the last topic in this discussion of design criteria is not meant to minimize its importance. Too often, designers either willingly or inadvertently sacrifice these objectives to mitigate other design concerns. Then, after design and in the midst of construction value engineering or schedule pressures, a subsequent design change may lead to conditions that will promote poor façade performance.

From the beginning of the project design process, designers should seek the input of construction experts, steel fabricators and erectors, and façade designers, manufacturers, and installers. Appropriate measures for constructability and economy may be highly dependent on such conditions as the project location, market trends, façade materials and systems, and schedule.

While it is true that constructability cannot be achieved without sufficient adjustability and clearance, the reverse is not true. Fabricators and erectors have seen many examples of details with creative means for adjustment that cannot be built economically. Hence, it is important that constructability discussions take place early in the design process.

Chapter 3
Overview of Responsibilities for Façade Attachment

Successful projects have clear lines of responsibility and effective communication between the responsible parties. This is especially true for façades and façade attachments because the strategy for façade attachment is influenced by the many disciplines involved in the project.

Most often, the architect prepares the contract documents for the building envelope and serves as the design professional of record for the façade elements. The architect sets the stage by selecting the façade system, or systems, for the project and setting many system parameters that affect the attachments. Often, design of the façade and its attachments is not within the purview of the structural engineer of record (SER), yet even then the SER must understand the strategy selected in order to design the primary building structure accordingly.

The façade structure and its attachments may be designed by a specialty structural engineer (SSE) working for the façade contractor, and the SSE may not become involved until after the structural construction documents are finalized. In addition, the façade system is often a prime target for last-minute value engineering to reduce project cost. The façade erector will further influence the design based on fabrication and erection needs.

Thus, the design of the façade system and its attachments requires integration and coordination of multiple parties throughout the project.

This chapter provides an overview of the responsibilities for each member of the project team with respect to façade attachments. These attachments, by definition, are at the interface between the structure and the façade and, as such, are at the boundary between the scope of work of the SER and the SSE designing the façade. Close coordination between the SER and the SSE will be instrumental in making the project flow smoothly.

Common practices for each façade trade vary and are reflected in the general overview presented here. The chapters on specific façade systems provide some of these trade-specific practices for division of responsibility.

3.1 THE OWNER'S RESPONSIBILITIES

The owner contributes to the requirements for the façade system, including the criteria for aesthetics, performance, and budget. This indirectly affects the façade attachments. Additionally, the owner ultimately controls the contractual relationship, either directly or indirectly, between the other parties of the project team. The owner is responsible for façade maintenance, which also may affect the attachments. In addition, some municipal regulations require that owners must have qualified professionals perform inspections of the façade at periodic intervals.

3.2 THE ARCHITECT'S RESPONSIBILITIES

The architect, or prime design professional (PDP) if the design team is not led by an architect, works with the owner to select the façade system that meets the requirements of the project. The façade system, along with other fundamental building design decisions, such as floor-to-floor heights, number of stories, and fenestration, influences the façade support and attachment strategy, which the architect selects with consultation from the SER and façade consultants, if any. The architect may also consult with the construction manager, façade manufacturers, and façade contractors as the fabrication and erection methods and schedule may influence the selection of the support and attachment strategies.

The architect's contract documents should define the attachment concepts and the relationship of the attachments to the façade components, building finishes, building systems, and structural components (the SER needs this information to ensure that the structural frame will accommodate the attachment strategy, or additional design work and coordination will be required at the time the contractor makes submittals). The architect specifies the design criteria for the façade, including design responsibilities, prescriptive or performance requirements, quality assurance requirements, and fabrication and erection tolerances. The attachment concepts shown on the drawings must be consistent with these specifications.

The architect also specifies the submittal and approval process for the façade, which includes the attachments.

3.3 THE SER'S RESPONSIBILITIES

The SER should consult with the architect on selecting façade support strategies and attachment concepts that are sensitive to the effects on the steel frame. Additionally, the SER provides the architect with information about anticipated structural movements as necessary to develop the façade specifications.

The SER designs the steel frame and slab edge conditions consistent with the façade support strategy and attachment concepts. The structural drawings should delineate the structural steel elements from the attachment elements to be

designed by the SSE. The structural drawings should also show the assumptions and limitations of the locations and magnitudes of the façade attachment loads.

The SER should show the fabrication and erection tolerances for the structural steel on the structural drawings and specifications. The structural steel details and concepts must account for sufficient field adjustment to accommodate the difference between the tolerances of the steel frame and the specified acceptable tolerance for the final façade position.

The SER reviews submittals by the SSE and the façade contractor specifically for the effect of the façade and its attachments on the primary building structure. The SER ensures that the design of the attachments is consistent with the loads associated with the support strategy and attachment concepts the SER used to design the steel framing.

The SER may or may not contract to assist the architect with development of portions of the façade design criteria, such as loads. If part of the SER's scope of services with the architect, the SER reviews the façade submittals for conformance with the structural design criteria for the façade.

3.4 THE SSE'S RESPONSIBILITIES

The SSE is the design professional responsible for the design of the façade and/or its attachments to the structural frame. The SSE for the attachments is usually under contract with the façade contractor. This is almost always the case if the SSE for the attachments is also the SSE for the design of the façade itself, as is often the case with precast suppliers, for example.

The SSE designs the attachments in accordance with the project specifications and consistent with the concepts and limitations presented in the design documents. This includes providing attachments with sufficient adjustability and clearance to meet the project requirements. The SSE prepares calculations and drawings for submittal in accordance with the project specifications. The SSE is responsible for the design of the attachments, including ensuring proper quality assurance for the installation in accordance with the requirements in the Applicable Building Code and project specifications.

When the Applicable Building Code requires special inspection, the SSE is responsible for inspecting the attachments as delegated by the SER.

3.5 THE GENERAL CONTRACTOR'S AND CONSTRUCTION MANAGER'S RESPONSIBILITIES

The general contractor coordinates the trades involved with the façade and its attachments. This usually includes the trades for the structural steel, concrete slab, and façade system(s). The general contractor coordinates the submittals of the various subcontractors and reviews them for coordination and conformance with the project specifications. The general contractor also ensures inspectors for the façade attachments have timely access to the work.

3.6 THE FAÇADE CONTRACTOR'S RESPONSIBILITIES

In addition to fabrication and erection of the façade, the façade contractor is often required by project specifications to be responsible for the structural design of the façade and its attachment. The SSE usually will be employed by either the general contractor or the façade contractor.

Chapter 4
Accommodating Construction Tolerances and Clearances in the Façade Attachment

Two or more materials meet at the building enclosure: the steel framing and the façade materials. Each has generally accepted industry-standard tolerances for their manufacture, fabrication, and erection or installation. The absolute value of the tolerances for steel framing is generally large in a relative sense when compared with the tolerances for the façade. Accordingly, adjustability must be provided between the structural details and façade attachment details to achieve a façade erected within acceptable tolerance relative to the theoretical plane.

Designers must also account for necessary clearances between the steel frame and the façade materials. The clearance is the space provided between the final adjusted location of the frame and/or slab component and the façade element to accommodate the tolerances and relative movements. Clearance may also be required for thermal insulation, fire safing or fireproofing elements, or access to install and adjust the façade component.

Designers can specify special steel frame tolerances to limit the amount of tolerance that must be addressed in the design and detailing of the façade systems and attachments. However, it is unreasonable to disregard the realities of construction practices. The AISC *Code of Standard Practice* (AISC, 2005a) defines tolerances for the steel frame that have been developed through long-standing usage as practical criteria. Although many frames are erected well within these tolerances, requiring more stringent tolerances should only be considered for exceptional situations, and based on consultation with fabricators and erectors local to the project. In the event that this is attempted and the design otherwise has no recourse to accommodate greater tolerances, there is a real risk to the project, as the probability of exceeding these tolerances is significant and the consequences to the project schedule and budget can be severe. Alternatively, the components of the wall will have to follow the frame, and this may compromise the appearance. Fortunately, a balance can be achieved by providing for the tolerances in the configuration of the attachment details.

4.1 TYPES OF TOLERANCES

Sources of dimensional variation include material production tolerances, fabrication and assembly tolerances, and erection and installation tolerances. Taken together, these comprise accumulated tolerances.

Material Production Tolerances

These are the allowable variations in the manufacture of the basic materials; examples include the allowable sweep in a steel beam or the allowable deviation in thickness of an architectural concrete panel. They are referred to as mill tolerances for steel and product tolerances for façade elements.

Fabrication and Assembly Tolerances

These are the allowable variations of subassemblies made from the basic material components; examples include the length tolerance on a fabricated column and the squareness of a preassembled panel in a unitized aluminum curtain wall.

Erection and Installation Tolerances

These are the allowable variations of installed components from their theoretical lines of location and plumbness; examples include the plumbness of erected steel columns and the allowable offsets of the outside face of adjacent façade panels.

Accumulated Tolerances

This is the total effect of the foregoing tolerances. Note that AISC *Code of Standard Practice* Section 7.12 limits the accumulation of mill and fabrication tolerances for steel frames to not exceed the erection tolerances.

Although unlikely, it is possible that the tolerances of each assembled component will vary to the maximum value allowed and combine in the same direction. Statistical research on the distribution of variations for each component and each source is not readily available to the designer. This information would be required for the designer to quantify the risk of exceeding a selected accumulated tolerance. Therefore, the designer does not know how the sum of the specified tolerances compares with the distribution of actual accumulated variations. Although it is not uncommon for designers to detail for an amount of accumulated variation that is less than the sum of the tolerances, they do so with an undetermined amount of risk.

A square root of the sum of the squares (SRSS) approach is one method designers use to combine the individual tolerances (Ballast, 1994) that results in a number less

(sometimes significantly less) than the absolute sum. There is no basis in statistical theory for this. For the case of façades supported on steel frames, the absolute value of tolerance of the steel frame is large compared with material, fabrication, and erection tolerances of the other components. This dominates the total for which designers must account. Thus, using an SRSS approach essentially means designers are optimistically assuming that the required adjustability to account for accumulated tolerance will not be much greater than that needed for the frame alone.

Designers must use judgment and understand the consequences when selecting an attachment strategy that cannot accommodate the accumulated tolerances based on the sum of the maximums for the individual components. Given the difference in magnitude between the tolerances for frame erection relative to tolerances for other components, designers should, as a minimum, account for the entire frame erection tolerance and use judgment for what additional amount is warranted. Alternatively, designers can consider the effect on the appearance and performance of the wall if it were to follow the as-built line of the steel frame. This is often the necessary approach for tall buildings.

4.2 STRUCTURAL STEEL TOLERANCES

Structural steel tolerances are composed of mill, fabrication, and erection tolerances. Provisions for adjustability will also be discussed.

Mill Tolerances

Designers should recognize the variations in cross-sectional geometry of wide-flange and other shapes. Figure 4-1 shows the mill tolerances on the cross-section of a W-shape. ASTM A6/A6M sets forth allowable variations in cross-section, as well as such length-based tolerances such as camber and sweep. For flange widths less than 6 in., camber variation (in.) is limited to the member length, *L* (ft), divided by 80. Sweep variation (in.) is limited to *L*/40. For flange widths equal to or greater than 6 in., the camber and sweep variations (in.) are both limited to *L*/80. Permitted variations of camber and sweep in column sections are specified according to length and section size. A convenient summary of tolerances for structural shapes based upon ASTM requirements is provided in Tables 1-22 through 1-29 in the AISC *Manual*.

Fabrication Tolerances

Section 6.4 in the AISC *Code of Standard Practice* defines the fabrication tolerances for the steel frame.

Erection Tolerances

Section 7.13 in the AISC *Code of Standard Practice* defines the erection tolerances for the steel frame. Figures 4-2 through 4-6 illustrate many of these tolerances.

Cumulative Tolerances

Of most interest to the designer on the subject of façade attachments is the necessary design adjustment for the total cumulative tolerance of the exterior frame. In particular, the adjustment must accommodate the allowable variation, inward or outward, of the frame line from the theoretical plumb column line at any floor elevation. Section 7.12 in the AISC *Code of Standard Practice* limits the accumulation of mill and fabrication tolerances for steel frames.

Tolerances for column base location, exterior column plumbness, and beam sweep are the most significant components of the cumulative tolerance relative to façade systems. Exterior column plumbness depends on the height above the base and the number of stories. The allowable spandrel beam sweep depends on the beam flange width and the beam length. Tables 4-1 through 4-6 provide the inward and outward (horizontal) tolerance at each floor for buildings up to 50 stories with combinations of 10-ft, 12-ft, and 14-ft story heights and 30-ft and 40-ft spandrel spans.

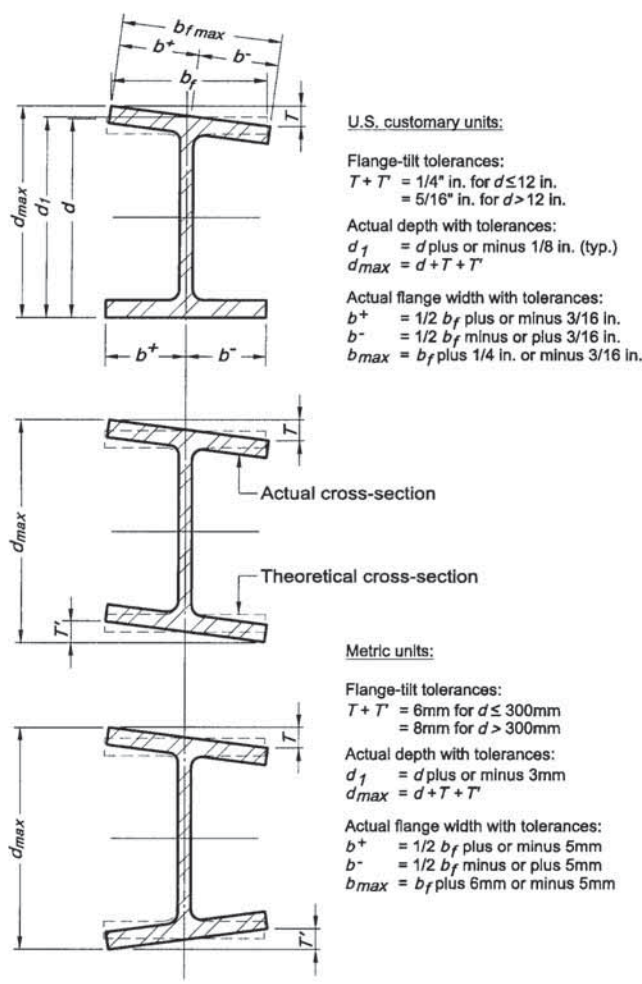


Fig. 4-1. Mill tolerances on the cross-section of W-shape (AISC, 2005a).

Table 4-1. Horizontal Tolerances for Steel Frames 10-ft Story Height and 30-ft Spandrel Span

Story	Δ_{csp}, in.		Δ_{cpt}, in.		Δ_{bsw}, in.		Δ_{tot}, in.	
	−	+	−	+	−	+	−	+
foundation	0.25	0.25	–	–	–	–	–	–
1	–	–	0.49	0.49	0.38	0.38	0.87	0.87
2	–	–	0.73	0.73	0.38	0.38	1.11	1.11
3	–	–	0.97	0.97	0.38	0.38	1.35	1.35
4	–	–	1.21	1.00	0.38	0.38	1.59	1.38
5	–	–	1.45	1.00	0.38	0.38	1.83	1.38
6	–	–	1.69	1.00	0.38	0.38	2.07	1.38
7	–	–	1.93	1.00	0.38	0.38	2.31	1.38
8	–	–	2.00	1.00	0.38	0.38	2.38	1.38
9	–	–	2.00	1.00	0.38	0.38	2.38	1.38
10	–	–	2.00	1.00	0.38	0.38	2.38	1.38
11	–	–	2.00	1.00	0.38	0.38	2.38	1.38
12	–	–	2.00	1.00	0.38	0.38	2.38	1.38
13	–	–	2.00	1.00	0.38	0.38	2.38	1.38
14	–	–	2.00	1.00	0.38	0.38	2.38	1.38
15	–	–	2.00	1.00	0.38	0.38	2.38	1.38
16	–	–	2.00	1.00	0.38	0.38	2.38	1.38
17	–	–	2.00	1.00	0.38	0.38	2.38	1.38
18	–	–	2.00	1.00	0.38	0.38	2.38	1.38
19	–	–	2.00	1.00	0.38	0.38	2.38	1.38
20	–	–	2.00	1.00	0.38	0.38	2.38	1.38
21	–	–	2.06	1.06	0.38	0.38	2.44	1.44
22	–	–	2.13	1.13	0.38	0.38	2.50	1.50
23	–	–	2.19	1.19	0.38	0.38	2.56	1.56
24	–	–	2.25	1.25	0.38	0.38	2.63	1.63
25	–	–	2.31	1.31	0.38	0.38	2.69	1.69
26	–	–	2.38	1.38	0.38	0.38	2.75	1.75
27	–	–	2.44	1.44	0.38	0.38	2.81	1.81
28	–	–	2.50	1.50	0.38	0.38	2.88	1.88
29	–	–	2.56	1.56	0.38	0.38	2.94	1.94
30	–	–	2.63	1.63	0.38	0.38	3.00	2.00
31	–	–	2.69	1.69	0.38	0.38	3.06	2.06
32	–	–	2.75	1.75	0.38	0.38	3.13	2.13
33	–	–	2.81	1.81	0.38	0.38	3.19	2.19
34	–	–	2.88	1.88	0.38	0.38	3.25	2.25
35	–	–	2.94	1.94	0.38	0.38	3.31	2.31
36	–	–	3.00	2.00	0.38	0.38	3.38	2.38
37	–	–	3.00	2.00	0.38	0.38	3.38	2.38
38	–	–	3.00	2.00	0.38	0.38	3.38	2.38
39	–	–	3.00	2.00	0.38	0.38	3.38	2.38
40	–	–	3.00	2.00	0.38	0.38	3.38	2.38
41	–	–	3.00	2.00	0.38	0.38	3.38	2.38
42	–	–	3.00	2.00	0.38	0.38	3.38	2.38
43	–	–	3.00	2.00	0.38	0.38	3.38	2.38
44	–	–	3.00	2.00	0.38	0.38	3.38	2.38
45	–	–	3.00	2.00	0.38	0.38	3.38	2.38
46	–	–	3.00	2.00	0.38	0.38	3.38	2.38
47	–	–	3.00	2.00	0.38	0.38	3.38	2.38
48	–	–	3.00	2.00	0.38	0.38	3.38	2.38
49	–	–	3.00	2.00	0.38	0.38	3.38	2.38
50	–	–	3.00	2.00	0.38	0.38	3.38	2.38

Notes:
1. Δ_{csp} = column base location tolerance
2. Δ_{cpt} = column plumbness tolerance (includes Δ_{csp})
3. Δ_{bsw} = beam/edge plate sweep tolerance
4. $\Delta_{tot}(+) = \Delta_{cpt}(+) + \Delta_{bsw}(+)$
5. $\Delta_{tot}(–) = \Delta_{cpt}(–) + \Delta_{bsw}(–)$
6. (+) = toward building line
(–) = away from building line

Table 4-2. Horizontal Tolerances for Steel Frames 10-ft Story Height and 40-ft Spandrel Span								
Story	Δ_{csp}, in.		Δ_{cpt}, in.		Δ_{bsw}, in.		Δ_{tot}, in.	
	–	+	–	+	–	+	–	+
foundation	0.25	0.25	–	–	–	–	–	–
1	–	–	0.49	0.49	0.50	0.50	0.99	0.99
2	–	–	0.73	0.73	0.50	0.50	1.23	1.23
3	–	–	0.97	0.97	0.50	0.50	1.47	1.47
4	–	–	1.21	1.00	0.50	0.50	1.71	1.50
5	–	–	1.45	1.00	0.50	0.50	1.95	1.50
6	–	–	1.69	1.00	0.50	0.50	2.19	1.50
7	–	–	1.93	1.00	0.50	0.50	2.43	1.50
8	–	–	2.00	1.00	0.50	0.50	2.50	1.50
9	–	–	2.00	1.00	0.50	0.50	2.50	1.50
10	–	–	2.00	1.00	0.50	0.50	2.50	1.50
11	–	–	2.00	1.00	0.50	0.50	2.50	1.50
12	–	–	2.00	1.00	0.50	0.50	2.50	1.50
13	–	–	2.00	1.00	0.50	0.50	2.50	1.50
14	–	–	2.00	1.00	0.50	0.50	2.50	1.50
15	–	–	2.00	1.00	0.50	0.50	2.50	1.50
16	–	–	2.00	1.00	0.50	0.50	2.50	1.50
17	–	–	2.00	1.00	0.50	0.50	2.50	1.50
18	–	–	2.00	1.00	0.50	0.50	2.50	1.50
19	–	–	2.00	1.00	0.50	0.50	2.50	1.50
20	–	–	2.00	1.00	0.50	0.50	2.50	1.50
21	–	–	2.06	1.06	0.50	0.50	2.56	1.56
22	–	–	2.13	1.13	0.50	0.50	2.63	1.63
23	–	–	2.19	1.19	0.50	0.50	2.69	1.69
24	–	–	2.25	1.25	0.50	0.50	2.75	1.75
25	–	–	2.31	1.31	0.50	0.50	2.81	1.81
26	–	–	2.38	1.38	0.50	0.50	2.88	1.88
27	–	–	2.44	1.44	0.50	0.50	2.94	1.94
28	–	–	2.50	1.50	0.50	0.50	3.00	2.00
29	–	–	2.56	1.56	0.50	0.50	3.06	2.06
30	–	–	2.63	1.63	0.50	0.50	3.13	2.13
31	–	–	2.69	1.69	0.50	0.50	3.19	2.19
32	–	–	2.75	1.75	0.50	0.50	3.25	2.25
33	–	–	2.81	1.81	0.50	0.50	3.31	2.31
34	–	–	2.88	1.88	0.50	0.50	3.38	2.38
35	–	–	2.94	1.94	0.50	0.50	3.44	2.44
36	–	–	3.00	2.00	0.50	0.50	3.50	2.50
37	–	–	3.00	2.00	0.50	0.50	3.50	2.50
38	–	–	3.00	2.00	0.50	0.50	3.50	2.50
39	–	–	3.00	2.00	0.50	0.50	3.50	2.50
40	–	–	3.00	2.00	0.50	0.50	3.50	2.50
41	–	–	3.00	2.00	0.50	0.50	3.50	2.50
42	–	–	3.00	2.00	0.50	0.50	3.50	2.50
43	–	–	3.00	2.00	0.50	0.50	3.50	2.50
44	–	–	3.00	2.00	0.50	0.50	3.50	2.50
45	–	–	3.00	2.00	0.50	0.50	3.50	2.50
46	–	–	3.00	2.00	0.50	0.50	3.50	2.50
47	–	–	3.00	2.00	0.50	0.50	3.50	2.50
48	–	–	3.00	2.00	0.50	0.50	3.50	2.50
49	–	–	3.00	2.00	0.50	0.50	3.50	2.50
50	–	–	3.00	2.00	0.50	0.50	3.50	2.50

Notes:
1. Δ_{csp} = column base location tolerance
2. Δ_{cpt} = column plumbness tolerance (includes Δ_{csp})
3. Δ_{bsw} = beam/edge plate sweep tolerance
4. $\Delta_{tot}(+) = \Delta_{cpt}(+) + \Delta_{bsw}(+)$
5. $\Delta_{tot}(–) = \Delta_{cpt}(–) + \Delta_{bsw}(–)$
6. (+) = toward building line
(–) = away from building line

Table 4-3. Horizontal Tolerances for Steel Frames 12-ft Story Height and 30-ft Spandrel Span								
Story	Δ_{csp}, in.		Δ_{cpt}, in.		Δ_{bsw}, in.		Δ_{tot}, in.	
	–	+	–	+	–	+	–	+
foundation	0.25	0.25	–	–	–	–	–	–
1	–	–	0.54	0.54	0.38	0.38	0.91	0.91
2	–	–	0.83	0.83	0.38	0.38	1.20	1.20
3	–	–	1.11	1.00	0.38	0.38	1.49	1.38
4	–	–	1.40	1.00	0.38	0.38	1.78	1.38
5	–	–	1.69	1.00	0.38	0.38	2.07	1.38
6	–	–	1.98	1.00	0.38	0.38	2.35	1.38
7	–	–	2.00	1.00	0.38	0.38	2.38	1.38
8	–	–	2.00	1.00	0.38	0.38	2.38	1.38
9	–	–	2.00	1.00	0.38	0.38	2.38	1.38
10	–	–	2.00	1.00	0.38	0.38	2.38	1.38
11	–	–	2.00	1.00	0.38	0.38	2.38	1.38
12	–	–	2.00	1.00	0.38	0.38	2.38	1.38
13	–	–	2.00	1.00	0.38	0.38	2.38	1.38
14	–	–	2.00	1.00	0.38	0.38	2.38	1.38
15	–	–	2.00	1.00	0.38	0.38	2.38	1.38
16	–	–	2.00	1.00	0.38	0.38	2.38	1.38
17	–	–	2.00	1.00	0.38	0.38	2.38	1.38
18	–	–	2.00	1.00	0.38	0.38	2.38	1.38
19	–	–	2.00	1.00	0.38	0.38	2.38	1.38
20	–	–	2.00	1.00	0.38	0.38	2.38	1.38
21	–	–	2.06	1.06	0.38	0.38	2.44	1.44
22	–	–	2.13	1.13	0.38	0.38	2.50	1.50
23	–	–	2.19	1.19	0.38	0.38	2.56	1.56
24	–	–	2.25	1.25	0.38	0.38	2.63	1.63
25	–	–	2.31	1.31	0.38	0.38	2.69	1.69
26	–	–	2.38	1.38	0.38	0.38	2.75	1.75
27	–	–	2.44	1.44	0.38	0.38	2.81	1.81
28	–	–	2.50	1.50	0.38	0.38	2.88	1.88
29	–	–	2.56	1.56	0.38	0.38	2.94	1.94
30	–	–	2.63	1.63	0.38	0.38	3.00	2.00
31	–	–	2.69	1.69	0.38	0.38	3.06	2.06
32	–	–	2.75	1.75	0.38	0.38	3.13	2.13
33	–	–	2.81	1.81	0.38	0.38	3.19	2.19
34	–	–	2.88	1.88	0.38	0.38	3.25	2.25
35	–	–	2.94	1.94	0.38	0.38	3.31	2.31
36	–	–	3.00	2.00	0.38	0.38	3.38	2.38
37	–	–	3.00	2.00	0.38	0.38	3.38	2.38
38	–	–	3.00	2.00	0.38	0.38	3.38	2.38
39	–	–	3.00	2.00	0.38	0.38	3.38	2.38
40	–	–	3.00	2.00	0.38	0.38	3.38	2.38
41	–	–	3.00	2.00	0.38	0.38	3.38	2.38
42	–	–	3.00	2.00	0.38	0.38	3.38	2.38
43	–	–	3.00	2.00	0.38	0.38	3.38	2.38
44	–	–	3.00	2.00	0.38	0.38	3.38	2.38
45	–	–	3.00	2.00	0.38	0.38	3.38	2.38
46	–	–	3.00	2.00	0.38	0.38	3.38	2.38
47	–	–	3.00	2.00	0.38	0.38	3.38	2.38
48	–	–	3.00	2.00	0.38	0.38	3.38	2.38
49	–	–	3.00	2.00	0.38	0.38	3.38	2.38
50	–	–	3.00	2.00	0.38	0.38	3.38	2.38

Notes:
1. Δ_{csp} = column base location tolerance
2. Δ_{cpt} = column plumbness tolerance (includes Δ_{csp})
3. Δ_{bsw} = beam/edge plate sweep tolerance
4. $\Delta_{tot}(+) = \Delta_{cpt}(+) + \Delta_{bsw}(+)$
5. $\Delta_{tot}(–) = \Delta_{cpt}(–) + \Delta_{bsw}(–)$
6. (+) = toward building line
(–) = away from building line

Table 4-4. Horizontal Tolerances for Steel Frames 12-ft Story Height and 40-ft Spandrel Span

Story	Δ_{csp}, in.		Δ_{cpt}, in.		Δ_{bsw}, in.		Δ_{tot}, in.	
	–	+	–	+	–	+	–	+
foundation	0.25	0.25	–	–	–	–	–	–
1	–	–	0.54	0.54	0.50	0.50	1.04	1.04
2	–	–	0.83	0.83	0.50	0.50	1.33	1.33
3	–	–	1.11	1.00	0.50	0.50	1.61	1.50
4	–	–	1.40	1.00	0.50	0.50	1.90	1.50
5	–	–	1.69	1.00	0.50	0.50	2.19	1.50
6	–	–	1.98	1.00	0.50	0.50	2.48	1.50
7	–	–	2.00	1.00	0.50	0.50	2.50	1.50
8	–	–	2.00	1.00	0.50	0.50	2.50	1.50
9	–	–	2.00	1.00	0.50	0.50	2.50	1.50
10	–	–	2.00	1.00	0.50	0.50	2.50	1.50
11	–	–	2.00	1.00	0.50	0.50	2.50	1.50
12	–	–	2.00	1.00	0.50	0.50	2.50	1.50
13	–	–	2.00	1.00	0.50	0.50	2.50	1.50
14	–	–	2.00	1.00	0.50	0.50	2.50	1.50
15	–	–	2.00	1.00	0.50	0.50	2.50	1.50
16	–	–	2.00	1.00	0.50	0.50	2.50	1.50
17	–	–	2.00	1.00	0.50	0.50	2.50	1.50
18	–	–	2.00	1.00	0.50	0.50	2.50	1.50
19	–	–	2.00	1.00	0.50	0.50	2.50	1.50
20	–	–	2.00	1.00	0.50	0.50	2.50	1.50
21	–	–	2.06	1.06	0.50	0.50	2.56	1.56
22	–	–	2.13	1.13	0.50	0.50	2.63	1.63
23	–	–	2.19	1.19	0.50	0.50	2.69	1.69
24	–	–	2.25	1.25	0.50	0.50	2.75	1.75
25	–	–	2.31	1.31	0.50	0.50	2.81	1.81
26	–	–	2.38	1.38	0.50	0.50	2.88	1.88
27	–	–	2.44	1.44	0.50	0.50	2.94	1.94
28	–	–	2.50	1.50	0.50	0.50	3.00	2.00
29	–	–	2.56	1.56	0.50	0.50	3.06	2.06
30	–	–	2.63	1.63	0.50	0.50	3.13	2.13
31	–	–	2.69	1.69	0.50	0.50	3.19	2.19
32	–	–	2.75	1.75	0.50	0.50	3.25	2.25
33	–	–	2.81	1.81	0.50	0.50	3.31	2.31
34	–	–	2.88	1.88	0.50	0.50	3.38	2.38
35	–	–	2.94	1.94	0.50	0.50	3.44	2.44
36	–	–	3.00	2.00	0.50	0.50	3.50	2.50
37	–	–	3.00	2.00	0.50	0.50	3.50	2.50
38	–	–	3.00	2.00	0.50	0.50	3.50	2.50
39	–	–	3.00	2.00	0.50	0.50	3.50	2.50
40	–	–	3.00	2.00	0.50	0.50	3.50	2.50
41	–	–	3.00	2.00	0.50	0.50	3.50	2.50
42	–	–	3.00	2.00	0.50	0.50	3.50	2.50
43	–	–	3.00	2.00	0.50	0.50	3.50	2.50
44	–	–	3.00	2.00	0.50	0.50	3.50	2.50
45	–	–	3.00	2.00	0.50	0.50	3.50	2.50
46	–	–	3.00	2.00	0.50	0.50	3.50	2.50
47	–	–	3.00	2.00	0.50	0.50	3.50	2.50
48	–	–	3.00	2.00	0.50	0.50	3.50	2.50
49	–	–	3.00	2.00	0.50	0.50	3.50	2.50
50	–	–	3.00	2.00	0.50	0.50	3.50	2.50

Notes:
1. Δ_{csp} = column base location tolerance
2. Δ_{cpt} = column plumbness tolerance (includes Δ_{csp})
3. Δ_{bsw} = beam/edge plate sweep tolerance
4. $\Delta_{tot}(+) = \Delta_{cpt}(+) + \Delta_{bsw}(+)$
5. $\Delta_{tot}(-) = \Delta_{cpt}(-) + \Delta_{bsw}(-)$
6. (+) = toward building line
(–) = away from building line

Table 4-5. Horizontal Tolerances for Steel Frames 14-ft Story Height and 30-ft Spandrel Span								
Story	Δ_{csp}, in.		Δ_{cpt}, in.		Δ_{bsw}, in.		Δ_{tot}, in.	
	−	+	−	+	−	+	−	+
foundation	0.25	0.25	–	–	–	–	–	–
1	–	–	0.59	0.59	0.38	0.38	0.96	0.96
2	–	–	0.92	0.92	0.38	0.38	1.30	1.30
3	–	–	1.26	1.00	0.38	0.38	1.63	1.38
4	–	–	1.59	1.00	0.38	0.38	1.97	1.38
5	–	–	1.93	1.00	0.38	0.38	2.31	1.38
6	–	–	2.00	1.00	0.38	0.38	2.38	1.38
7	–	–	2.00	1.00	0.38	0.38	2.38	1.38
8	–	–	2.00	1.00	0.38	0.38	2.38	1.38
9	–	–	2.00	1.00	0.38	0.38	2.38	1.38
10	–	–	2.00	1.00	0.38	0.38	2.38	1.38
11	–	–	2.00	1.00	0.38	0.38	2.38	1.38
12	–	–	2.00	1.00	0.38	0.38	2.38	1.38
13	–	–	2.00	1.00	0.38	0.38	2.38	1.38
14	–	–	2.00	1.00	0.38	0.38	2.38	1.38
15	–	–	2.00	1.00	0.38	0.38	2.38	1.38
16	–	–	2.00	1.00	0.38	0.38	2.38	1.38
17	–	–	2.00	1.00	0.38	0.38	2.38	1.38
18	–	–	2.00	1.00	0.38	0.38	2.38	1.38
19	–	–	2.00	1.00	0.38	0.38	2.38	1.38
20	–	–	2.00	1.00	0.38	0.38	2.38	1.38
21	–	–	2.06	1.06	0.38	0.38	2.44	1.44
22	–	–	2.13	1.13	0.38	0.38	2.50	1.50
23	–	–	2.19	1.19	0.38	0.38	2.56	1.56
24	–	–	2.25	1.25	0.38	0.38	2.63	1.63
25	–	–	2.31	1.31	0.38	0.38	2.69	1.69
26	–	–	2.38	1.38	0.38	0.38	2.75	1.75
27	–	–	2.44	1.44	0.38	0.38	2.81	1.81
28	–	–	2.50	1.50	0.38	0.38	2.88	1.88
29	–	–	2.56	1.56	0.38	0.38	2.94	1.94
30	–	–	2.63	1.63	0.38	0.38	3.00	2.00
31	–	–	2.69	1.69	0.38	0.38	3.06	2.06
32	–	–	2.75	1.75	0.38	0.38	3.13	2.13
33	–	–	2.81	1.81	0.38	0.38	3.19	2.19
34	–	–	2.88	1.88	0.38	0.38	3.25	2.25
35	–	–	2.94	1.94	0.38	0.38	3.31	2.31
36	–	–	3.00	2.00	0.38	0.38	3.38	2.38
37	–	–	3.00	2.00	0.38	0.38	3.38	2.38
38	–	–	3.00	2.00	0.38	0.38	3.38	2.38
39	–	–	3.00	2.00	0.38	0.38	3.38	2.38
40	–	–	3.00	2.00	0.38	0.38	3.38	2.38
41	–	–	3.00	2.00	0.38	0.38	3.38	2.38
42	–	–	3.00	2.00	0.38	0.38	3.38	2.38
43	–	–	3.00	2.00	0.38	0.38	3.38	2.38
44	–	–	3.00	2.00	0.38	0.38	3.38	2.38
45	–	–	3.00	2.00	0.38	0.38	3.38	2.38
46	–	–	3.00	2.00	0.38	0.38	3.38	2.38
47	–	–	3.00	2.00	0.38	0.38	3.38	2.38
48	–	–	3.00	2.00	0.38	0.38	3.38	2.38
49	–	–	3.00	2.00	0.38	0.38	3.38	2.38
50	–	–	3.00	2.00	0.38	0.38	3.38	2.38

Notes:
1. Δ_{csp} = column base location tolerance
2. Δ_{cpt} = column plumbness tolerance (includes Δ_{csp})
3. Δ_{bsw} = beam/edge plate sweep tolerance
4. $\Delta_{tot}(+) = \Delta_{cpt}(+) + \Delta_{bsw}(+)$
5. $\Delta_{tot}(-) = \Delta_{cpt}(-) + \Delta_{bsw}(-)$
6. (+) = toward building line
(–) = away from building line

Table 4-6. Horizontal Tolerances for Steel Frames 14-ft Story Height and 40-ft Spandrel Span								
Story	Δ_{csp}, in.		Δ_{cpt}, in.		Δ_{bsw}, in.		Δ_{tot}, in.	
	–	+	–	+	–	+	–	+
foundation	0.25	0.25	–	–	–	–	–	–
1	–	–	0.59	0.59	0.50	0.50	1.09	1.09
2	–	–	0.92	0.92	0.50	0.50	1.42	1.42
3	–	–	1.26	1.00	0.50	0.50	1.76	1.50
4	–	–	1.59	1.00	0.50	0.50	2.09	1.50
5	–	–	1.93	1.00	0.50	0.50	2.43	1.50
6	–	–	2.00	1.00	0.50	0.50	2.50	1.50
7	–	–	2.00	1.00	0.50	0.50	2.50	1.50
8	–	–	2.00	1.00	0.50	0.50	2.50	1.50
9	–	–	2.00	1.00	0.50	0.50	2.50	1.50
10	–	–	2.00	1.00	0.50	0.50	2.50	1.50
11	–	–	2.00	1.00	0.50	0.50	2.50	1.50
12	–	–	2.00	1.00	0.50	0.50	2.50	1.50
13	–	–	2.00	1.00	0.50	0.50	2.50	1.50
14	–	–	2.00	1.00	0.50	0.50	2.50	1.50
15	–	–	2.00	1.00	0.50	0.50	2.50	1.50
16	–	–	2.00	1.00	0.50	0.50	2.50	1.50
17	–	–	2.00	1.00	0.50	0.50	2.50	1.50
18	–	–	2.00	1.00	0.50	0.50	2.50	1.50
19	–	–	2.00	1.00	0.50	0.50	2.50	1.50
20	–	–	2.00	1.00	0.50	0.50	2.50	1.50
21	–	–	2.06	1.06	0.50	0.50	2.56	1.56
22	–	–	2.13	1.13	0.50	0.50	2.63	1.63
23	–	–	2.19	1.19	0.50	0.50	2.69	1.69
24	–	–	2.25	1.25	0.50	0.50	2.75	1.75
25	–	–	2.31	1.31	0.50	0.50	2.81	1.81
26	–	–	2.38	1.38	0.50	0.50	2.88	1.88
27	–	–	2.44	1.44	0.50	0.50	2.94	1.94
28	–	–	2.50	1.50	0.50	0.50	3.00	2.00
29	–	–	2.56	1.56	0.50	0.50	3.06	2.06
30	–	–	2.63	1.63	0.50	0.50	3.13	2.13
31	–	–	2.69	1.69	0.50	0.50	3.19	2.19
32	–	–	2.75	1.75	0.50	0.50	3.25	2.25
33	–	–	2.81	1.81	0.50	0.50	3.31	2.31
34	–	–	2.88	1.88	0.50	0.50	3.38	2.38
35	–	–	2.94	1.94	0.50	0.50	3.44	2.44
36	–	–	3.00	2.00	0.50	0.50	3.50	2.50
37	–	–	3.00	2.00	0.50	0.50	3.50	2.50
38	–	–	3.00	2.00	0.50	0.50	3.50	2.50
39	–	–	3.00	2.00	0.50	0.50	3.50	2.50
40	–	–	3.00	2.00	0.50	0.50	3.50	2.50
41	–	–	3.00	2.00	0.50	0.50	3.50	2.50
42	–	–	3.00	2.00	0.50	0.50	3.50	2.50
43	–	–	3.00	2.00	0.50	0.50	3.50	2.50
44	–	–	3.00	2.00	0.50	0.50	3.50	2.50
45	–	–	3.00	2.00	0.50	0.50	3.50	2.50
46	–	–	3.00	2.00	0.50	0.50	3.50	2.50
47	–	–	3.00	2.00	0.50	0.50	3.50	2.50
48	–	–	3.00	2.00	0.50	0.50	3.50	2.50
49	–	–	3.00	2.00	0.50	0.50	3.50	2.50
50	–	–	3.00	2.00	0.50	0.50	3.50	2.50

Notes:
1. Δ_{csp} = column base location tolerance
2. Δ_{cpt} = column plumbness tolerance (includes Δ_{csp})
3. Δ_{bsw} = beam/edge plate sweep tolerance
4. $\Delta_{tot}(+) = \Delta_{cpt}(+) + \Delta_{bsw}(+)$
5. $\Delta_{tot}(–) = \Delta_{cpt}(–) + \Delta_{bsw}(–)$
6. (+) = toward building line
   (–) = away from building line

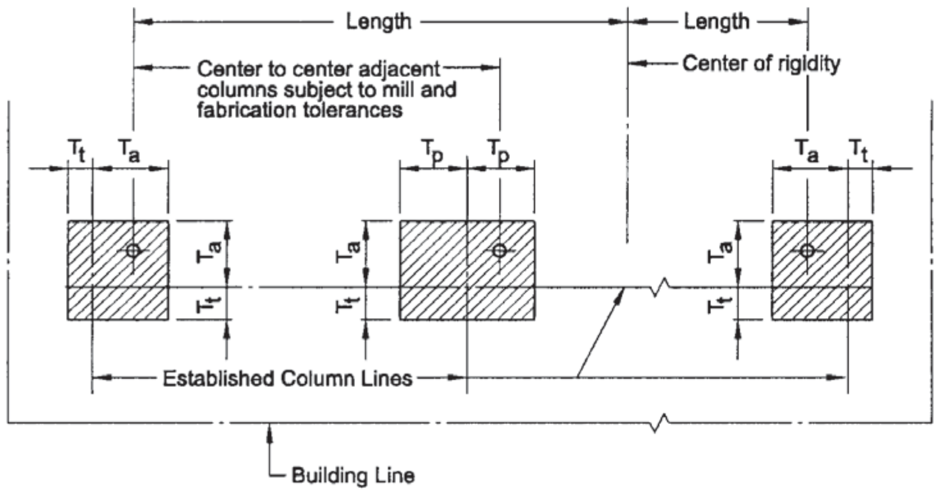


Fig. 4-2. Tolerances in plan location of column (AISC, 2005a).

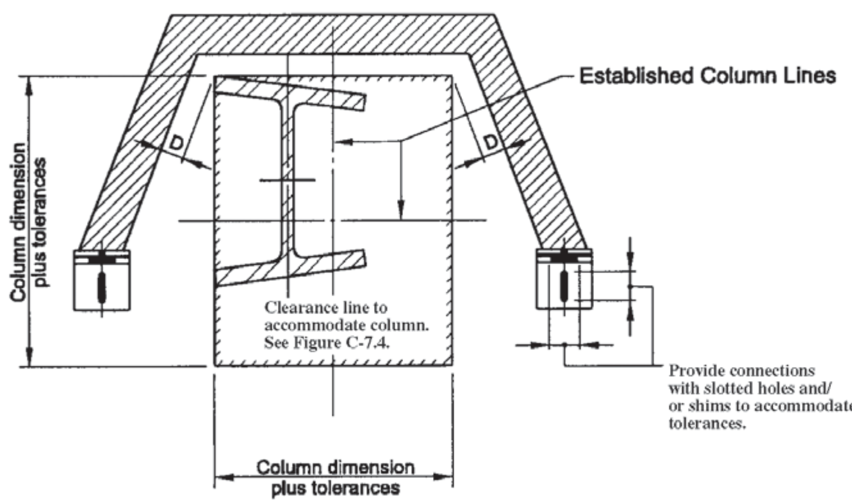


Fig. 4-3. Clearance required to accommodate fascia (AISC, 2005a).

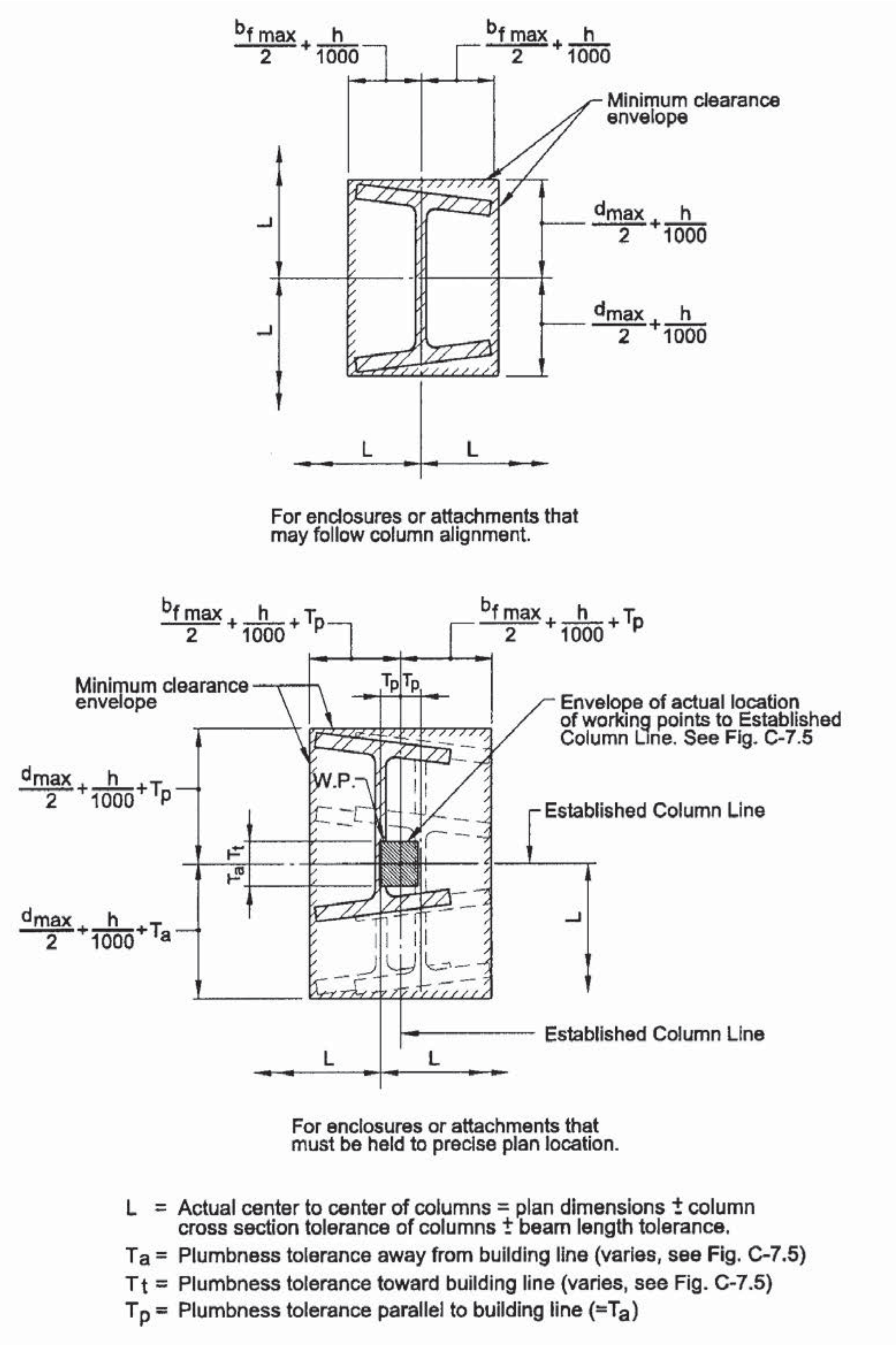


Fig. 4-4. Clearance required to accommodate accumulated column tolerance (AISC, 2005a).

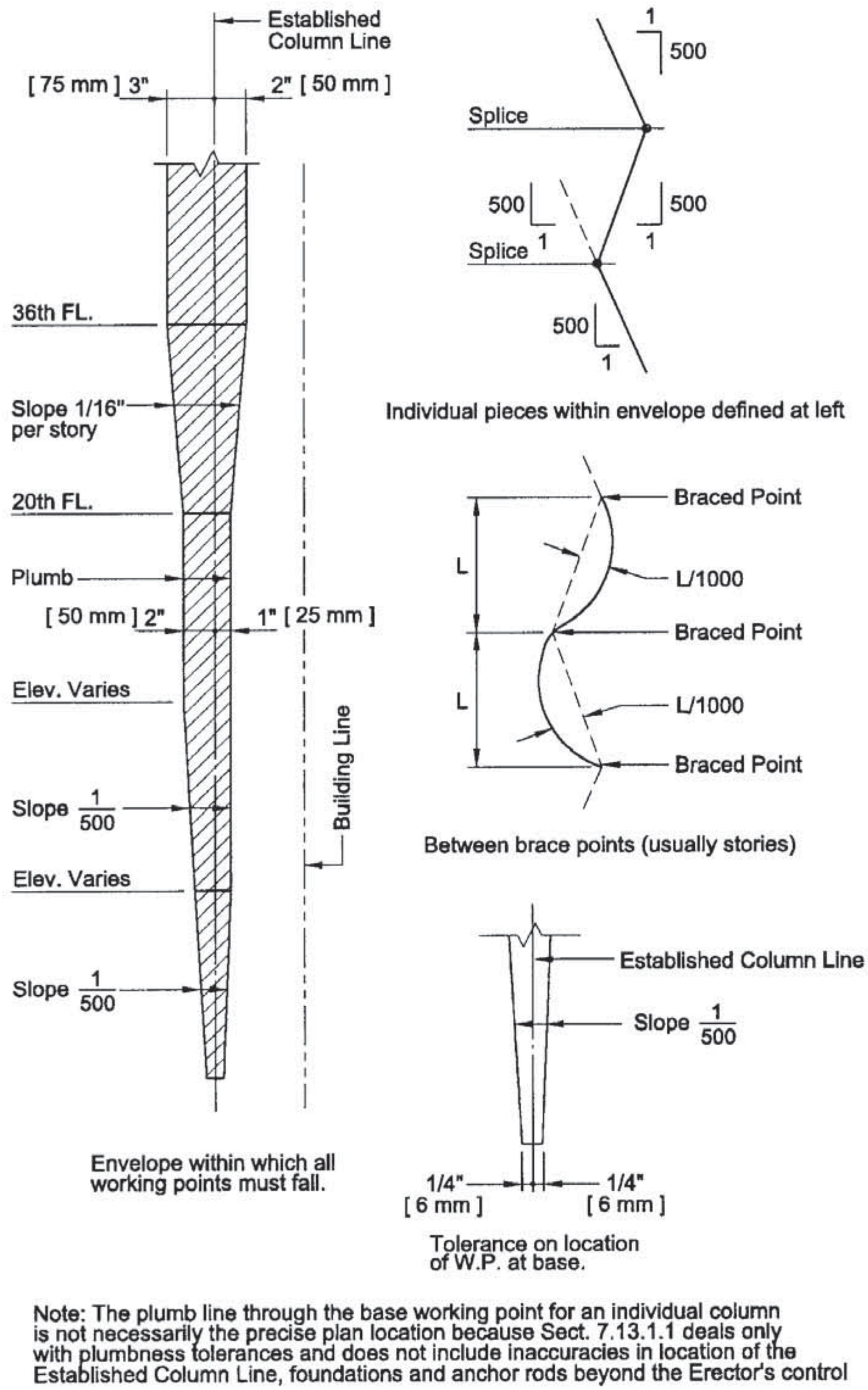


Fig. 4-5. Exterior column plumbness tolerances normal to building (AISC, 2005a).

The cumulative tolerance for the elevation of a point at mid-span of a spandrel beam is composed of the allowable variation of the beam work point down from the upper column splice line (5⁄16-in. up and 3⁄16-in. down from theoretical location) and the allowable variation in camber. The column splice line elevation is based on cumulative tolerances of the base plate elevation and fabricated member length tolerances. The maximum variation of the top of the base plate relative to the bearing surface is 1⁄8 in., up or down. The column fabrication tolerances vary depending on length and connectivity to other structural steel members, but are under no condition greater than 1⁄8 in.

Adjustable Items

As required in Section 7.13.1.3, adjustable items must be designated as adjustable in the contract documents. Thus, the designer should clearly identify in the contract documents items that are intended to be adjustable, such as shelf angles, slab edge elements, and similar supporting members for other trades, and provide the total adjustability required for proper alignment. Section 7.13.1.3 in the AISC *Code of Standard Practice* permits a variation for adjustable items of plus or minus 3⁄8 in. in both the vertical and horizontal directions.

The AISC *Code of Standard Practice* also requires that designers provide the necessary clearances and adjustments for materials furnished by other trades to accommodate the steel tolerances defined in the Code. Furthermore the Code requires the "Owner's Designated Representative for Construction" to check the final steel location prior to placing or installing façade elements.

4.3 FAÇADE MATERIAL AND ERECTION TOLERANCES

Numerous tolerances apply to the façade systems discussed in this Design Guide, however only those that relate most significantly to the attachment of façades to steel structures are outlined below. The references in which these tolerances are listed also include various product tolerances that may be helpful to designers of façade systems.

4.3.1 Brick Veneer Tolerances

Brick veneer assembly tolerances, as defined by ACI 530 (ACI et al., 2005), are relatively small compared to those for the structural frame. They are:

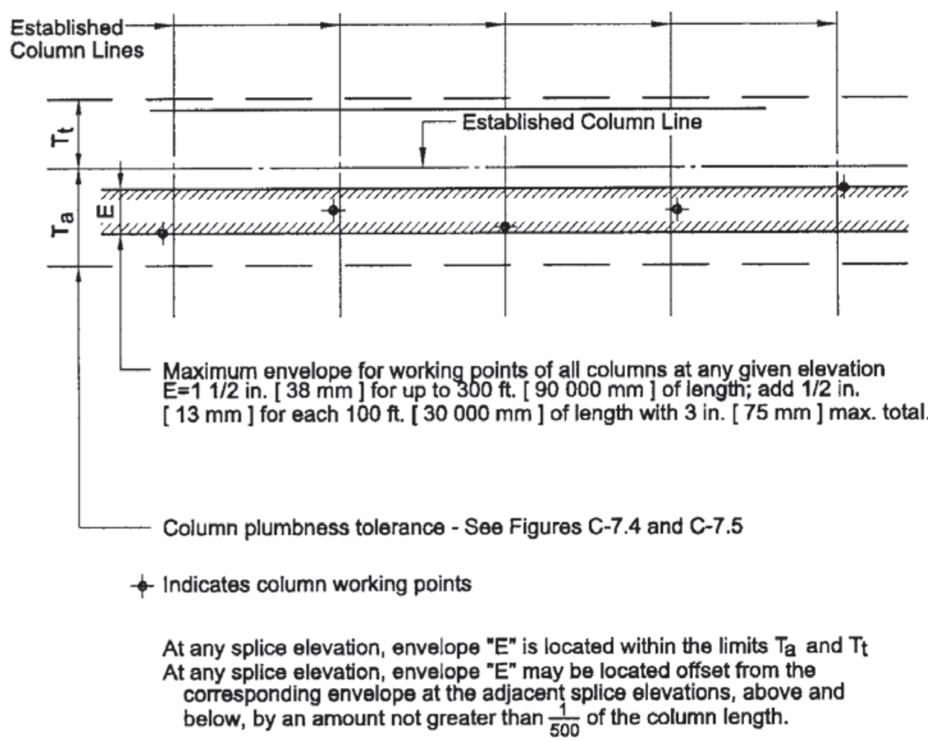


Fig. 4-6. Tolerances in plan at any splice elevation of exterior columns (AISC, 2005a).

Elements:

Variation from plumb

- ± ¼ in. in 10 ft
- ± ⅜ in. in 20 ft
- ± ½ in. max.

True to a line

- ± ¼ in. in 10 ft
- ± ⅜ in. in 20 ft
- ± ½ in. max.

Location of Elements:

Indicated in plan

- ± ½ in. in 20 ft
- ± ⅜ in. max.

Indicated in elevation

- ± ¼ in. in story height
- ± ¾ in. max.

4.3.2 Precast Concrete Panel Tolerances

The Precast/Prestressed Concrete Institute (PCI) provides its *Tolerance Manual for Precast and Prestressed Concrete Construction*, (PCI, 2000), which is widely referenced for product and erection tolerances. Additionally, precast-concrete panel tolerances are defined in the PCI *Design Handbook* (PCI, 2004) as follows:

Plan Location:

From building grid datum

- ± ½ in.

From centerline of steel support (takes precedence over dimension from grid datum)

- ± ½ in.

Support Elevation:

From nominal elevation

- – ½ in.
- + ¼ in.

Elements:

Variation from plumb (minimum of height of structure or 100 ft)

- ± 1 in.

Variation from plumb (in any 10 ft)

- ± ¼ in.

Warping variation from nearest adjacent corner

- ± ¹⁄₁₆ in. per ft

Bowing per panel

- *L*/360

Differential bowing of panels of the same design

- ½ in.

4.3.3 Aluminum Curtain Wall Tolerances

The American Architectural Manufacturers Association (AAMA) provides guidance in *Installation of Aluminum Curtain Walls* (AAMA, 1989a) and Metal Curtain Wall Manual (AAMA, 1989b), which specify tolerances for aluminum curtain walls as follows:

Elements:

Bowing standard mill tolerance for extruded aluminum

- ¼ in. in 20-ft length

Twisting standard mill tolerance for extruded aluminum

- 5° in 20-ft length

Location of Elements:

Variation from plane or location shown on approved shop drawings

- ⅛ in. per 12 ft
- ½ in. in any total length

Maximum offset from true alignment between two identical members abutting end to end in line

- ¹⁄₁₆ in.

The Construction Sciences Research Foundation (CSRF) provides its *SPECTEXT® Master Guide Specification, Section 08920* "Glazed Aluminum Curtain Wall" (CSRF, 2006), which recommends the following:

Elements:

Variation from plumb (minimum of, noncumulative)

- 0.06 in. per 3 ft
- ½ in. per 100 ft

Misalignment of adjacent members (noncumulative)

- $\frac{1}{32}$ in.

4.3.4 GFRC Panel Tolerances

GFRC panel tolerances as defined in PCI *Recommended Practice for Glass Fiber Reinforced Concrete Panels* (PCI, 2001) are as follows:

Plan Location:

From building grid datum

- ± ½ in.

From centerline of steel support (takes precedence over dimension from grid datum)

- ± ½ in.

Support Elevation:

From nominal elevation

- – ½ in.
- + ¼ in.

Elements:

Variation from plumb (minimum of height of structure or 100 ft)

- ± 1 in.

Variation from plumb (in any 10 ft)

- ± ¼ in.

Warping variation from nearest adjacent corner

- ± $\frac{1}{16}$ in. per ft

Bowing per panel

- *L*/240

Differential bowing of panels of the same design

- ¼ in.

4.3.5 EIFS Panel Tolerances

There are few standards for EIFS panel tolerances, but *Exterior Insulation and Finish Systems Design Handbook* (Thomas, 1992) suggests the following:

Elements:

Offset from surface plane of adjacent panels

- ± $\frac{1}{16}$ in. and not discernible from a distance of more than 6 ft when viewed perpendicular to panel faces

Corner to corner squareness

- ± ⅛ in.

Chapter 5
Design of Slab Edge Conditions for Façade Attachments

5.1 GENERAL APPROACHES

The slab edge detail is an important consideration when designing the façade attachments. Factors that influence the design approach include:

- The type of façade and its location relative to the steel frame;
- How far the slab and/or deck overhangs the spandrel beams;
- The strength of the slab or deck, and the degree to which the designer wishes to count on the slab or deck for façade attachment;
- Whether or not the façade loads are applied directly to the slab or deck; and,
- The degree to which the edge condition is similar throughout the project.

Of these, the degree to which the designer wishes to count on the slab or deck to support loads has the most influence on the approach and detail. There are two fundamental approaches for designing the slab edge. First, the slab or deck can be cantilevered over the spandrel beam to carry any loads applied to the slab or deck overhang. Alternatively, and second, the designer can ignore the slab or deck as a cantilever and apply the loads to the overhang with a steel plate or assembly.

Generally speaking, counting on the slab or deck to carry the loads as a cantilever is the most economical approach if the typical slab or deck can provide the strength necessary for the given overhang and loads. It is usually not economical to increase the typical slab or deck thickness just for the overhang condition. When the overhang and/or loads are large relative to the slab strength, the designer may neglect the strength of the slab or deck and add a bent plate or other structural steel assembly attached to the spandrel beam to resist the loads at the slab edge.

There may be other project conditions that compel the designer to neglect the slab or deck strength as a cantilever. Some examples are:

- When there is no concrete slab and the metal deck is parallel with the spandrel beam;
- When slab depressions and openings in the back-span make counting on the cantilever problematic for too many locations;
- When the designer anticipates quality control issues with the concrete slab strength and placement; and,
- When the façade attachment details dictate a heavy bent plate for attachment anyway.

Figure 5-1 illustrates a slab designed as a cantilever overhanging the spandrel beam. The overhanging slab, as well as the back span, which is the first interior span of slab, must have shear and flexural strength sufficient for the loads. This approach has the added benefit of resolving the eccentricities of the slab edge loads without creating torsion in the spandrel beam. For these practical and economical reasons, the designer should include the slab and deck in the support system, when possible. A bent plate or light-gage metal pour stop is still needed until the concrete reaches adequate strength.

Façade attachment loads to the slab edge will generally include vertical forces, F_V, and horizontal forces, F_H. In addition, depending on the fixity of the connection of the attachment clips to the façade, the attachment forces may include a moment, M_{ef}, due to the eccentricity between the vertical forces at the center of gravity of the façade system and the edge of the slab.

Note that the horizontal force, F_H, is shown in Figure 5-1 as a tension force, indicating that the façade panel is hung from this support at its top. Were the panel instead supported at its bottom, F_H would be a compressive force.

Figure 5-2 illustrates the concepts where the SER designs a steel edge member, such as a bent plate, to transfer the loads at the slab edge to the spandrel. The slab and/or deck are not designed for shear or flexure from the overhang. However, either may be designed for any in-plane horizontal forces that the façade attachments may deliver to the slab. In this approach, the spandrel will have torsion applied to it due to the eccentricity between the spandrel web and the loads at the slab edge.

The applied torsion must be resolved by the support conditions of the spandrel, or another means. Given that wide-flange shapes are very economical, but not very efficient at resisting torsion, it is usually best to eliminate the torsion by framing beams or kickers into the spandrel at the locations

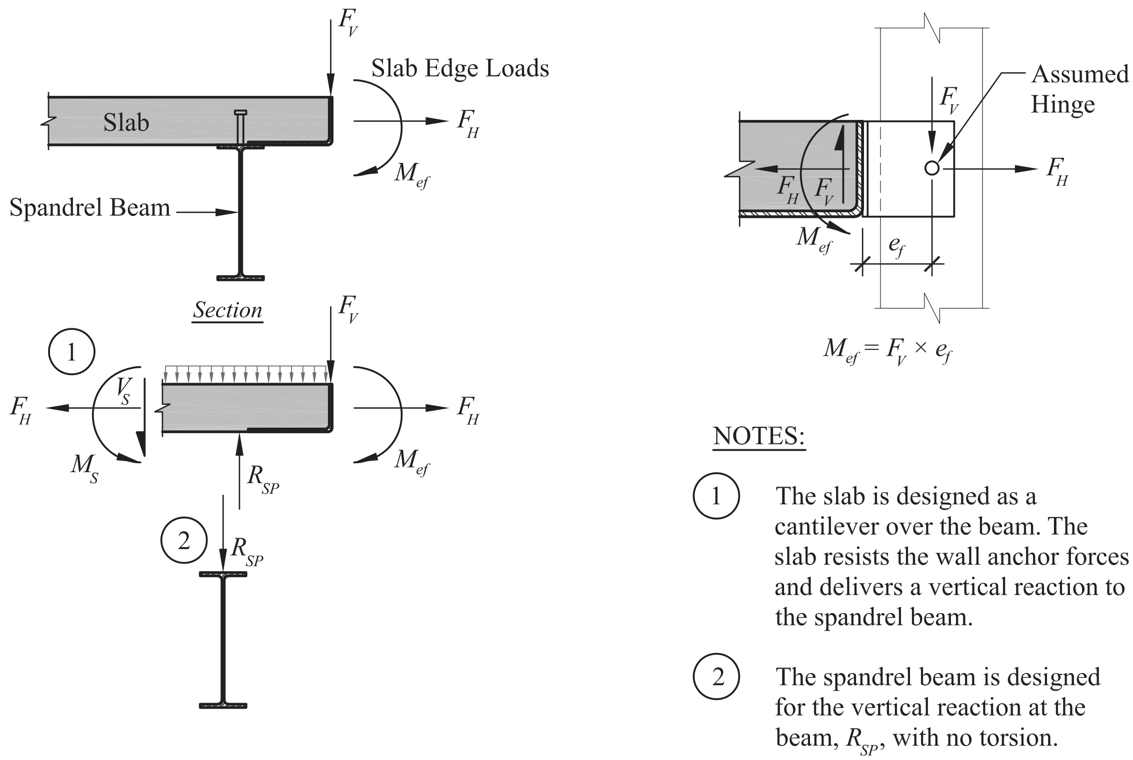


Fig. 5-1. Design concept where slab is used to resolve eccentricity of slab edge loads.

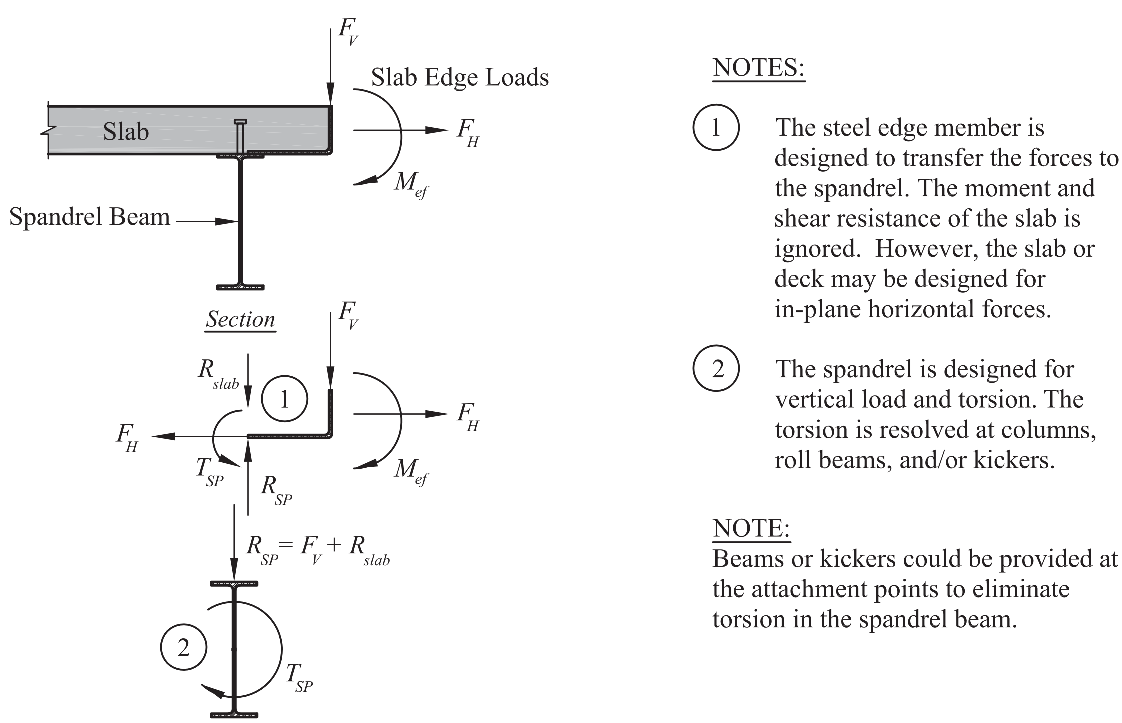


Fig. 5-2. Design concept where torsion in spandrel is used to resolve eccentricity of slab edge loads.

of the façade attachments to resist the moment and eliminate the torsion on the spandrel entirely. This requires coordination with the architecture and mechanical systems, which may limit the use of these solutions. If so, these elements can be used in acceptable locations to limit the length of the spandrel beam to restrain the twist.

When the spandrel is a girder, the floor beams spanning into the spandrel are usually capable of restraining the twist, provided that the connections to the girder are capable of transferring the moment and are not too flexible. When the floor beams are parallel to the spandrel, one option is to add sub-framing perpendicular to the spandrel between the spandrel and the first interior floor beam. These sub-beams are sometimes referred to as "roll beams." The other option is to add kickers between the bottom flange of the spandrel and either the top flange of an interior floor beam or an anchored connection to the slab. In both options, if the designer considers the spandrel connections to the columns as additional points of restraint, the design of the connection to the column must address torsion and flexibility. Figures 5-3 and 5-4 provide a closer look at the load path to restrain torsion in the spandrel beam using roll beams and kickers, respectively.

Whether the designer is counting on the slab to cantilever over the spandrel, or is designing the spandrel beam for torsion, successful design requires careful attention to the load path through the components and connections, as well as attention to those elements that contribute to the overall deflection of the slab edge. The following sections provide guidance for considering these issues when designing slab edge details.

5.2 SLAB EDGE DETAILS WITH LIGHT-GAGE METAL POUR STOPS

One of the most economical slab edge details incorporates a light-gage metal pour stop. Usually made of cold-formed steel of 10 to 20 gage, the pour stop is part of the metal decking procurement package and is welded to the spandrel beam in the field during erection of the metal deck. Figure 5-5 presents a typical light-gage metal pour stop detail.

As the name implies, the sole purpose of the pour stop is to form the edge of the concrete slab. As published by the Steel Deck Institute (SDI, 2000) and provided by many deck manufacturers in their catalogues, the design of the pour stop is for the weight of the wet concrete, concrete fluid pressure on the vertical leg, and a uniform construction live load of 20 psf. The SER must design the concrete slab to carry all superimposed dead and live loads after the concrete is cured. Therefore, the use of the light-gage metal pour stop is generally limited to the following conditions:

- The adjusted overhang, measured from the flange tip, is 12 in. or less;
- The configuration of the slab and its reinforcement is such that it has the strength to resist all superimposed loads on the overhang, including façade attachment loads, if any; and,
- If the overhang carries any façade attachment loads, the load path must be directly into the slab and not through the pour stop itself. The light-gage metal pour stop rarely has the strength required for façade attachments. Furthermore, if steel embedment plates are cast into the slab for façade attachment, only very heavy-gage pour stops are stiff enough to secure the embedment plates to them for positioning in the slab.

When using light-gage metal pour stops, the designer must provide for support of the pour stop at columns and corners of the slab edge.

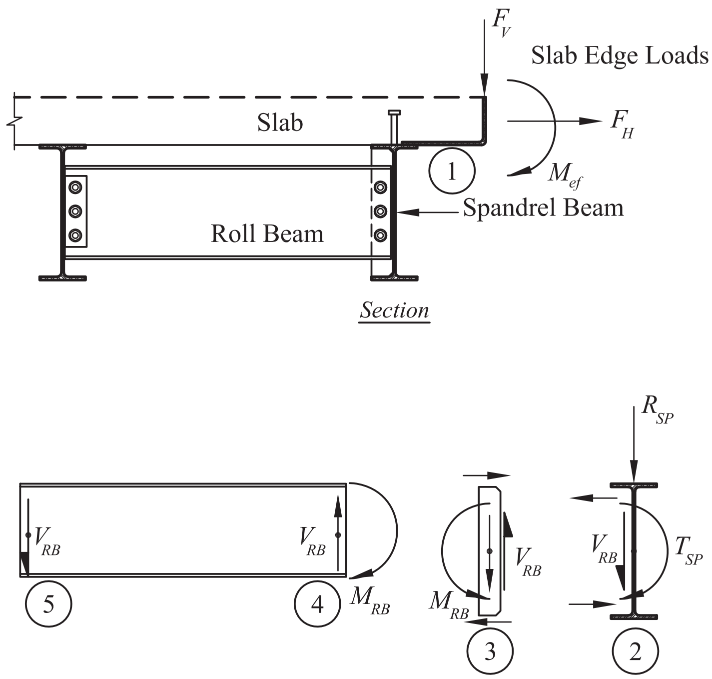
5.2.1 Design of Light-Gage Metal Pour Stops

Figure 5-6 outlines the design approach for the cold-formed, light-gage metal pour stop. The pour stop is designed as a cantilever with the critical moment location at the tip of the spandrel beam flange. The wet concrete loads include the wet weight of the concrete as a uniform load over the horizontal leg and an equivalent fluid pressure, equal to the wet weight of the concrete, applied to the vertical leg. An additional construction live load of 20 psf is applied as a uniform load over the horizontal leg. Generally, a concentrated construction live load is not considered for light-gage metal pour stops.

Common designs call for the pour stop to overlap the spandrel flange by 2 in. The back edge of the pour stop is welded to the top of the spandrel flange. Bearing at the tip of the spandrel flange is used to transmit the shear from the cantilever. A force-couple formed between this bearing force at the flange tip and an equal and opposite transverse force in the weld resolve the cantilever moment.

Designers can specify the pour stop as an adjustable item and specify the maximum and minimum overlap of the pour stop with the spandrel beam flange. This will allow field adjustment of the pour stop to address some of the steel frame tolerances. For instance, if the maximum anticipated adjustment at a particular spandrel is 1 in., and a minimum of 2 in. of bearing is required for the pour stop, the overlap can be specified as ranging between a minimum of 2 in. and a maximum of 4 in.

Light-gage metal pour stops are generally made from material with specified minimum yield strength, F_y, of 33 ksi. SDI recommends that the designer limit the design flexural stress to 20 ksi for the wet concrete load, temporarily increased by one-third for the additional construction live load. SDI also recommends limiting the horizontal and



NOTES:

1. The steel edge member is designed to transfer the forces to the spandrel. The slab is neglected, except for horizontal forces.

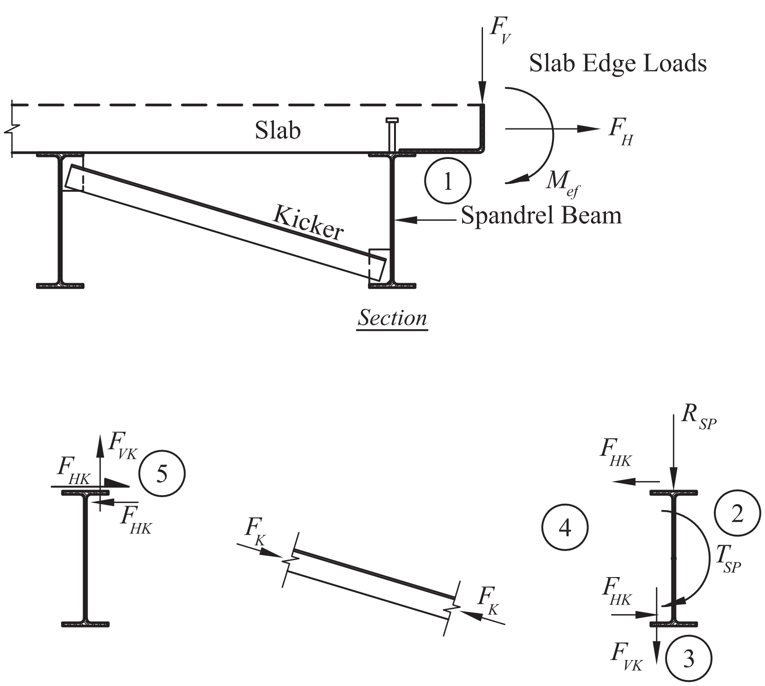
2. The spandrel beam is designed for the vertical reactions and torsion between roll beams. The roll beams restrain twist of the spandrel at intermittent locations.

3. A full-depth stiffener or other connection is designed for moment and resulting vertical reactions. The reactions may include floor loads if the roll beam is also a floor beam.

4. The moment in the roll beam is determined as the accumulated torsion from the spandrel.

5. The moment applied to the end of the roll beam is resolved by vertical shears at each end.

Fig. 5-3. Design concept using roll beam to resolve torsion in spandrel from slab edge loads.



NOTES:

1. The steel edge member is designed to transfer the forces to the spandrel. The slab is neglected.

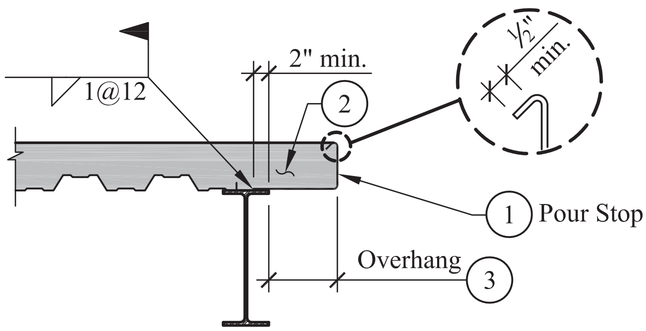
2. The spandrel beam is designed for the vertical reactions and torsion between the kicker locations.

3. The kicker reaction results in both horizontal and vertical load to the bottom flange of the spandrel.

4. The force-couple at the top and bottom flanges resolves accumulated torsion from the spandrel. The top flange of the spandrel must be restrained by the deck/slab or a supplemental member.

5. The kicker applies vertical and horizontal forces to an interior beam and/or slab.

Fig. 5-4. Design concept using kickers to resolve torsion in spandrel from slab edge loads.



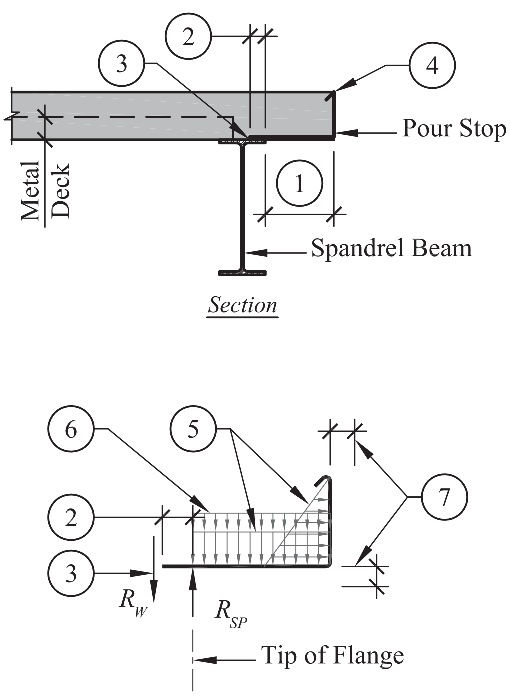
NOTES:

1. The pour stop is designed for the wet weight of concrete, concrete fluid pressure on the vertical leg, and a 20 psf live load.
2. The slab is designed for self weight and all superimposed loads.
3. The overhang is generally less than 12 in. for light-gage metal pour stops.

See Table 5-1 for additional notes.

Metal deck parallel to spandrel

Fig. 5-5. Light-gage metal pour stop detail.



NOTES:

1. The pour stop is designed as a cantilever overhanging the tip of the spandrel beam flange.
2. The overlap forms a back span for the cantilever. The specified overlap should equal the design overlap plus tolerance.
3. The pour stop is field welded to the spandrel flange. The weld strength required equals the overhang moment divided by the design overlap.
4. The vertical leg is returned to stiffen the top edge of the pour stop.
5. Design loads include the wet weight and equivalent fluid pressure of the concrete.
6. Design loads also include a construction live load of 20 psf.
7. Horizontal and vertical deflections are limited to ¼ in. for the wet concrete loads.

Fig. 5-6. Design approach for light-gage metal pour stop.

vertical deflection of the tip of the vertical leg to ¼ in. This guidance is incorporated into Table 5-1 (see end of chapter), which provides a pour stop selection table (used with permission of SDI).

Example 5.1 illustrates light-gage metal pour stop selection using this table.

5.2.2 Design of Slab Overhang Made with Light-Gage Metal Pour Stop for Superimposed Loads

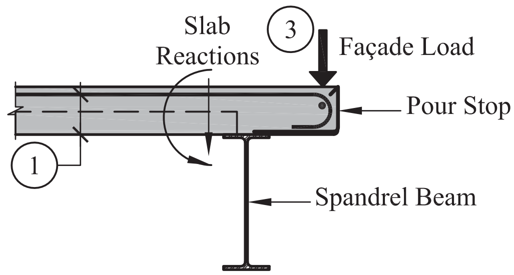
When using slab edge details made with a light-gage metal pour stop, designers must design the concrete slab to carry all superimposed dead and live loads after the concrete is cured. As a practical matter, the concrete weight generally is included, thereby making the pour stop a stay-in-place form not necessary for the performance of the cured slab.

The slab overhang is generally designed as a cantilever with the back span being formed by the span of the slab between the spandrel and the next interior beam or girder. The critical section for the design of the slab is usually on the back-span side of the spandrel due to the effective width or depth reductions from the flutes of the metal deck. Figure 5-7 illustrates the effective width and depth reductions. The most important of these reductions is the depth reduction that occurs on the back-span side when the flutes of the deck are parallel with the spandrel beam. It is not unusual for this to result in depths that are one half of the overhanging slab depth.

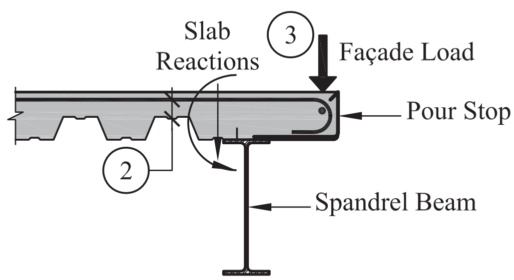
If the overhang is to carry any façade attachment loads, the load path to the slab should be through direct bearing. The light-gauge metal pour stop itself is ineffective as a receptor of welded façade attachments. If the façade attachments are applied to the slab overhang, they may very well be concentrated loads, and the designer must determine the effective width of the slab for checking the strength. A conservative but customary assumption is to assume a critical width equal to the actual width of the applied load plus two times the distance from the load to the section of interest, as illustrated in Figure 5-8.

Slab overhangs and back spans can be designed using ACI 318 (ACI, 2004) requirements for reinforced concrete slabs. If façade loads are applied to the slab edge on the overhang, these loads will usually control the design of the top reinforcement in the slab at these locations, and it is not unusual for designs to call for additional deformed bars in the top of the slab.

Tables 5-2 through 5-9 provide slab flexural strengths for various slab thicknesses, deck heights, deck orientations, and concrete strengths and slab reinforcement weights.



Metal deck perpendicular to spandrel



Metal deck parallel to spandrel

NOTES:

1 The full slab depth is effective when the deck is perpendicular to the spandrel. The effective width is reduced by the presence of the flutes in the deck.

2 Only the slab depth above the deck is effective when the deck is parallel to the spandrel. The effective width, however, is not reduced.

3 The façade load is transferred directly to the slab. The light-gage metal pour stop is not effective as a means of façade attachment.

Fig. 5-7. Effective depths of slabs on metal deck at slab edge.

Example 5.2 illustrates the design of a concrete slab overhanging a spandrel beam and reinforced to support façade loads.

5.3 SLAB EDGE DETAILS WITH STRUCTURAL STEEL BENT PLATES OR OTHER STEEL EDGE MEMBERS

There are at least three cases when designers might choose to use a bent plate or other structural steel element at the slab edge instead of a light-gage metal pour stop:

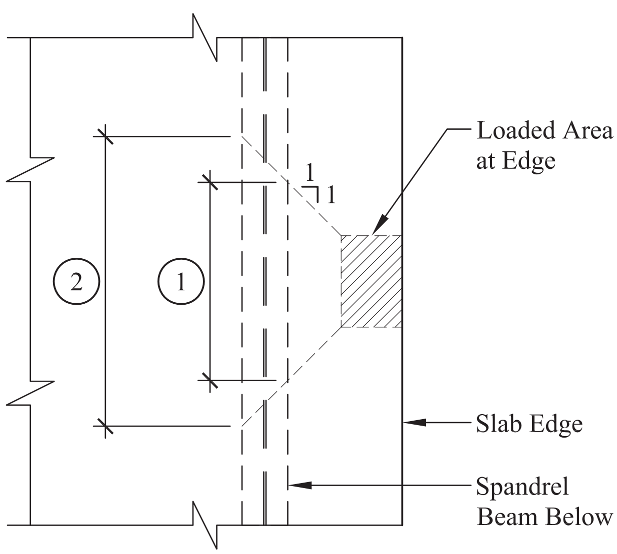
- Case 1—The overhang is larger than what is appropriate for a light-gage metal pour stop. In this case, the slab or deck can be used to carry any loads on the overhang and a bent plate can be used only as a pour stop. Façade attachments will be made directly to the slab, or to an embedment plate in the slab.
- Case 2—The same as Case 1, except a structural steel bent plate (or similar steel element) is used as a means to attach the façade elements to the slab. As one example, headed studs can be used on the vertical leg of the bent plate embedded in the slab with field-welded façade attachment clips attached to the plate after the slab is placed.
- Case 3—The slab and/or deck do not have adequate strength or stiffness for the overhang loads and a bent plate or other edge element will serve as the structural means to transmit the loads from the overhang back to the spandrel beam. As described earlier, this results in eccentric loads on the spandrel that the designer must address.

Case 3 generally occurs at roof framing without slabs, or at floor framing made of open web joists or otherwise incorporating thin slabs or metal decks where the concrete is just a fill and not considered a structural element. Section 5.1 provides other project conditions that might compel the designer to neglect the slab or deck strength as a cantilever.

A variation of Case 3 is the condition where the designer neglects the transverse shear and flexural strength of the slab overhang and designs the bent plate to carry these forces, but accounts for strength of the slab to take in-plane forces. In this case the designer can use a headed stud or deformed bar anchor to transfer the horizontal façade forces into the plane of the slab where any tension can be carried by the slab reinforcement.

A bent plate is often used in any of these cases. Minimum thicknesses are generally 3⁄16 in., and maximum thicknesses are generally ½ in., limited by the bending equipment capacity in the shop or field handling of the pieces if they are not shop attached to the spandrel beam. Designers should be aware that as thicknesses increase, the practical lengths of the plates get shorter. The limits are shop and erector specific, but it is not uncommon for ½-in. plates to be held to 10-ft lengths. Therefore, there may be three lengths of plate on a 30-ft spandrel beam, and designers must decide if a connection is required between such segments of plate along a single spandrel beam.

Hot-rolled angles have two advantages over the bent plate. First, they do not have the same length limitations. Second, their shape tolerances are much tighter; the angle between the legs is more consistently 90 degrees and the tips of the



NOTES:

1. Effective width at tip of flange at slab overhang.
2. Effective width at critical section for slab back span. Note that if the deck is perpendicular to the spandrel, the effective width is reduced by the presence of the flutes in the deck. Use the sum of the rib widths within the effective strip.

Fig. 5-8. Effective width for concentrated loads at slab edge.

outstanding legs are straighter. However, the necessary geometry for the detail is often such that the designer cannot choose an angle, either because the needed leg sizes do not exist or, if they do, the available thickness is much greater than needed. Additionally, the designer should consider rolling tolerances of angles including their sweep.

The bent plate or angle can be applied to the spandrel beam in the shop or field, or shop applied and field adjusted. By far, the most economical approach from the perspective of steel fabrication and erection is to simply shop-attach the bent plate or angle. However, tolerances and clearances for the façade must then be accounted for by other means, such as in the façade attachment clips. When designing a detail that allows shop attachment and field adjustment, the designer must consider the following:

- The needed outward and inward adjustment;
- The spandrel beam flange width, realizing that the most economical spandrel beam shape may vary throughout the project and thus change the flange width;
- The minimum overlap between the plate and flange needed for the connection;
- What the maximum overlap between the plate and flange may be and ensure this does not cause interference with other construction, such as installation of headed studs; and,
- The lengths of slotted holes and the hole edge distances to the tips of the flange and edges of the plate.

Figure 5-9 provides some example connection details for bent plates on spandrel beams.

As with light-gage metal pour stops, the designer must provide for support of the pour stop at columns and corners of the slab edge.

5.3.1 Case 1—Bent Plate Used as a Pour Stop Only

When the overhang is larger than what is appropriate for a light-gage metal pour stop, a bent plate can be used to form the edge of slab. If the slab is designed to carry the superimposed loads and the plate is not a means to attach the façade to the slab, its sole purpose is to serve as a pour stop, and the design of the plate is similar to that for the light-gage metal pour stop.

As with light-gage metal pour stops, the design of the bent plate pour stop is for the weight and lateral fluid pressure of the wet concrete, and for a uniform construction live load of 20 psf. Consider limiting the vertical and horizontal deflection of the tip of the up-turned leg to less than ⅛ in. when subjected to the wet concrete load (this deflection limit is a judgment and designers may choose another limit, such as ¼ in. as is used in the SDI table for light-gage metal pour stops). If the overhang gets larger than about 12 in., designers should consider also checking the plate for a concentrated construction load of at least 250 pounds at the tip in place of the uniform construction live load of 20 psf.

Tables 5-10 and 5-11 provide design aids for selecting bent plates for unstiffened pour stops.

Example 5.3 illustrates the design of a bent plate pour stop.

5.3.2 Case 2—Bent Plate Used as a Pour Stop and a Means of Attaching the Façade to the Slab

A situation where the designer may choose a bent plate over a light-gage metal pour stop is when the designer wishes to use the bent plate as a means to attach the façade to the slab. The slab is still reinforced to carry the superimposed loads, including façade loads, applied to the overhang. A field-applied headed stud or deformed bar anchor is usually welded to the vertical leg of the bent plate.

The bent plate may be a means to attach an embedment plate for the façade attachments. Sample details are shown in Figure 5-10. Headed studs and embedment plates that are located within or protrude into the width of the beam flange must be field welded to conform to the OSHA steel erection rules for tripping hazards.

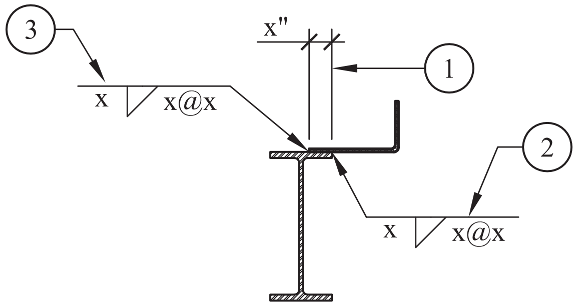
As discussed in Section 5.1, façade attachment loads to the slab edge will generally include vertical forces, F_V, and horizontal forces, F_H. In addition, depending on the fixity of the connection of the attachment clips to the façade, the attachment forces may include a moment, M_{ef}, due to the eccentricity between the vertical forces at the center of gravity of the façade system and the edge of the slab. Figure 5-11 illustrates the design concept where a headed stud on the vertical leg of a bent plate transfers the façade attachment forces to the slab. The headed stud (or deformed bar anchor if the designer so chooses) must be capable of transferring shear and tension to the concrete slab. The headed stud shear and pullout strengths must account for the edge distances that are present.

Tables 5-12 through 5-15 are design aids that provide shear and tension strength values for headed studs in the edges of concrete slabs.

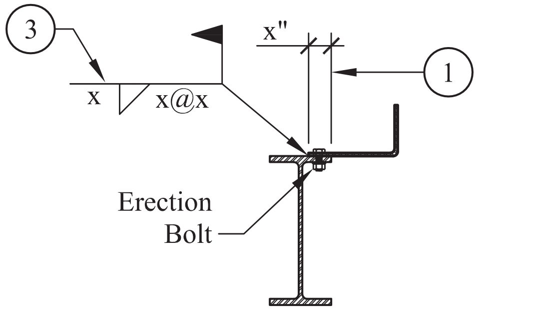
Example 5.4 illustrates the design of a bent plate pour stop that is also used as a means to attach the façade, with loads transmitted to slab reinforcement with headed studs.

5.3.3 Case 3—Bent Plate Transmits the Overhang Loads; the Slab Transverse Shear and Flexural Strength Are Neglected

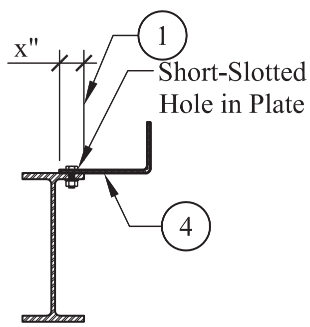
The SER sometimes cannot count on the slab or deck to carry the loads on the slab overhang because of depressions or penetrations in the back span. With long overhangs and/or large loads, such as precast-concrete panel weights, the bent



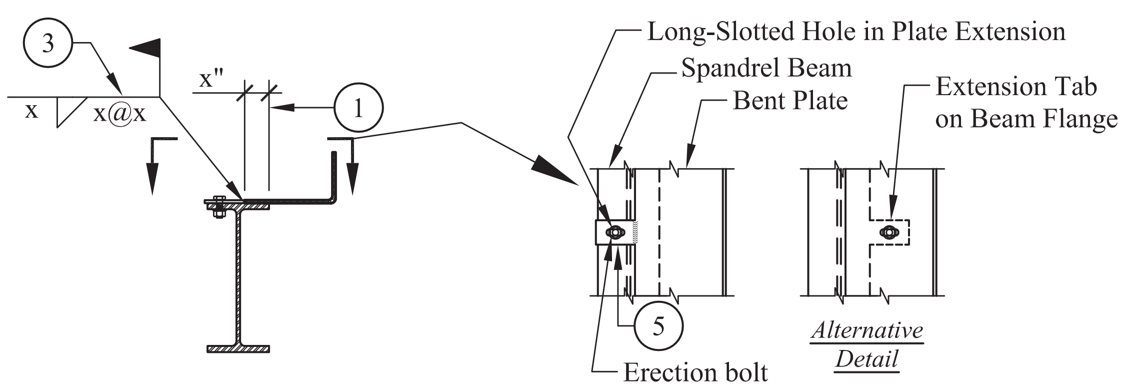
(a) Shop-welded (no field adjustment)



(b) Field-welded (some field adjustment)



(c) Shop- or field-bolted (minimal field adjustment)



(d) Shop- or field-bolted w/ field adjustment
(maximum field adjustment)

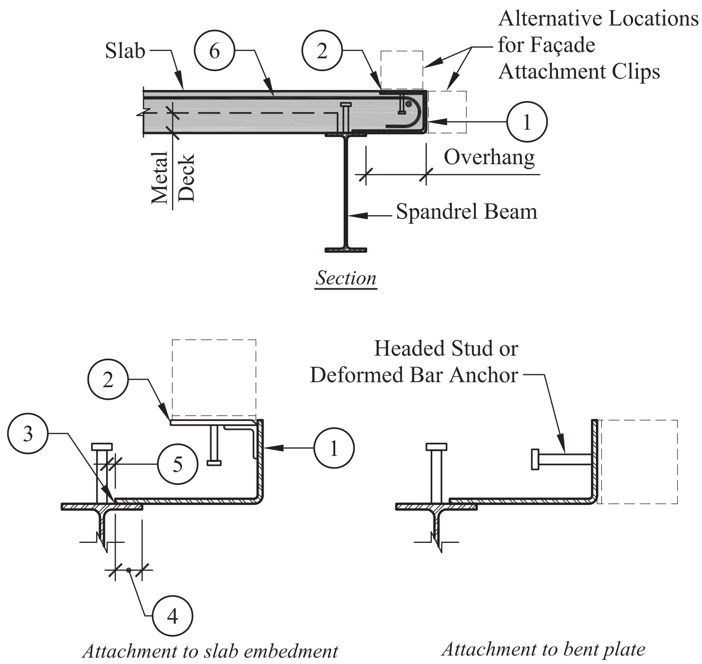
NOTES:

1. Indicate design overlap with minimum and maximum for tolerance and adjustment. Design should consider minimum flange widths for minimum overlap, minimum deck bearing, and room for headed studs.
2. Provide nominal amount of weld to resist incidental uplift during erection.
3. Provide weld as required to resist loads on plate.
4. Field adjustment is usually modest due to geometry of flange width and bolt hole edge distances.
5. Plate extensions and erection bolts provide for shop attachment with field adjustment. Alternatively, extension tabs can be attached to beam flange tip.

Fig. 5-9. Example details for connection of bent plate to spandrel beam.

plate, or other edge assembly, can be designed to transfer the overhang loads to the spandrel. This will inevitably apply some torsion to the spandrel and the designer must assess the impact of this on the spandrel design.

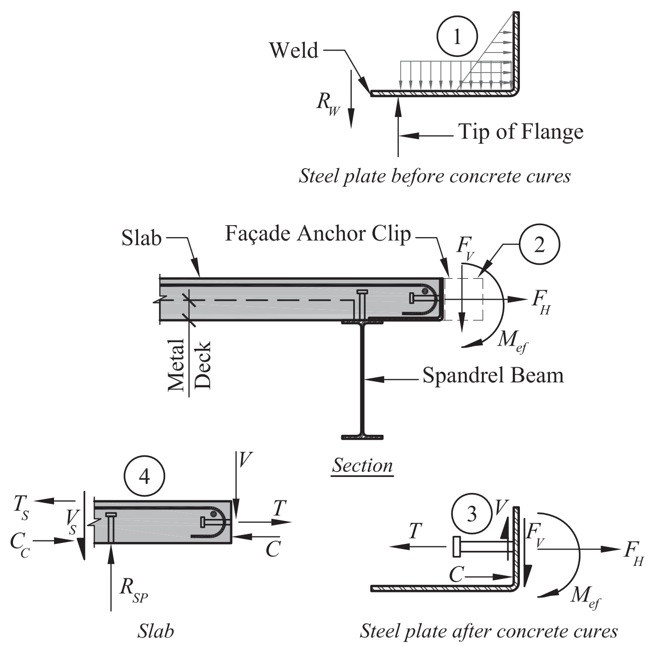
Although there may be conditions where the transverse shear and flexural strength of the slab overhang must be neglected, horizontal forces usually can be transferred to the slab with a headed stud or deformed bar anchor on the



NOTES:

1. Design bent plate as pour stop. Commonly $\frac{3}{16}$-in. min. and $\frac{1}{2}$-in. max. thickness. Consider bent plate rather than light-gage metal pour stop for large overhangs and/or embedment plates.
2. Example of embedment plate for facade attachment. Field weld or otherwise field anchor to steel closure plate. Design to transfer facade attachment loads to slab.
3. Shop weld pour stop to spandrel beam if tolerances are accomodated in the facade attachment details; otherwise field weld.
4. Overlap as necessary for weld design.
5. Maintain minimum clearance for headed stud installation (including stud ferrule).
6. Design slab to cantilever over spandrel and support loads from facade attachment.

Fig. 5-10. Typical detail of steel edge member as pour stop and façade attachment.



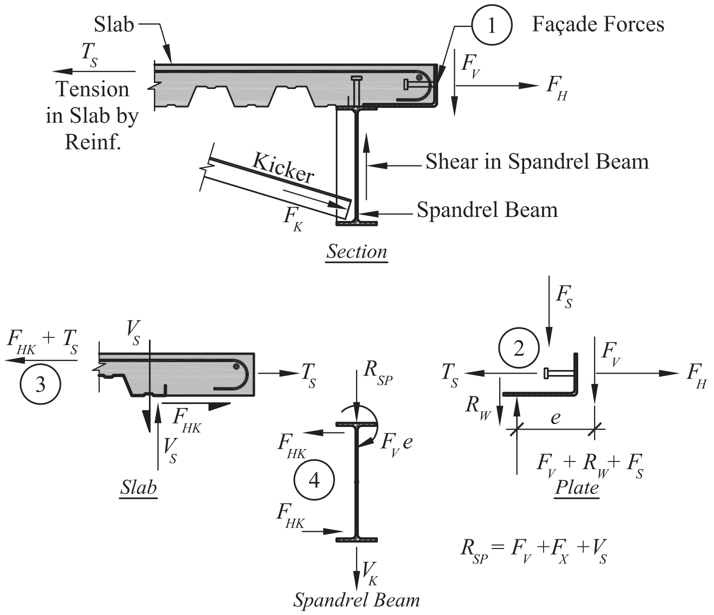
NOTES:

1. Steel edge assembly supports wet concrete and construction loads similar to light-gage metal pour stop.
2. After concrete cures, steel edge assembly provides means for anchoring facade to slab.
3. Assembly must transfer shear, moment, and horizontal force to slab with a headed stud or other means.
4. Reinforced concrete slab is designed to cantilever over spandrel.

Fig. 5-11. Concept where steel assembly forms pour stop and means to attach façade.

back of the vertical leg, thereby eliminating or reducing the bending in the bent plate. Figure 5-12 illustrates this concept. Note that there must be a load path for these in-plane forces to get to the main slab diaphragm of the structure. Figure 5-13 illustrates the transfer of the façade horizontal force to the slab.

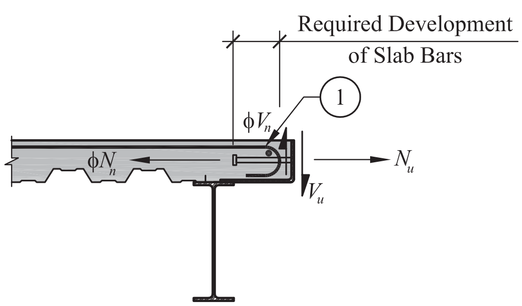
The length of the headed stud should be such that it extends into the slab a sufficient distance to allow transfer of the force



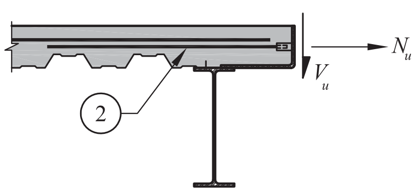
NOTES:

1. When only the slab's in-plane strength is considered, the steel plate (or assembly) must deliver the vertical load (F_V) to the base structure (spandrel beam).
2. The steel plate and headed stud (or deformed bar dowel) transfers the horizontal facade force to the slab.
3. The slab reinforcement is designed for the tension from the horizontal façade force plus the horizontal force from the couple resulting from the kicker.
4. The spandrel beam is designed for the vertical loads and reactions, plus torsion between the kickers if the façade loads are applied between kickers.

Fig. 5-12. Design concept for slab edge detail where slab is neglected except for in-plane forces.



(a) Headed stud welded to bent plate



(b) Dowel bar coupler welded to bent plate

NOTES:

1. Develop slab reinforcement with 180-degree hook within embedment of headed stud.
2. Deformed bar dowel threaded into bar coupler welded to bent plate. Lap with slab reinforcement.

Fig. 5-13. Transfer of horizontal façade force to the slab edge.

to the slab reinforcement. Slab bars with 180-degree hooks can be used to reduce the development length of the slab bars; when the resulting hook depth does not fit within the slab vertically, the bar can be rotated from vertical. If necessary, deformed bar dowels can be attached to the back of the plate with threaded couplers welded to the plate. This allows for longer lap lengths to develop the force into the slab.

For large overhangs and/or large loads, the required thickness of a bent plate may become impractical (it is the author's experience that ½ in. is the limit of practical bent plate thickness). Figure 5-14 shows slab edge details that can be used in such a case.

As the plate becomes thick and/or requires stiffeners for the applied façade loads, and the overhang is less than 12 in., the designer may want to provide separate brackets or other steel assemblies from the spandrel beam to support the façade loads and allow a light-gage metal pour stop to form the edge of slab.

Example 5.5 illustrates the design of a bent plate pour stop supporting a façade with the slab strength ignored.

Example 5.6 is similar to Example 5.5, except welded bar couplers and threaded reinforcing bars are used to transmit out-of-plane forces only to the slab.

Example 5.1—Light-Gage Metal Pour Stop Selection

For the light-gage metal pour stop illustrated in Figure 5-15, determine the required thickness based on the maximum slab overhang.

Given:
The slab is composed of normal-weight concrete (w_c = 150 lb/ft^3), and the spandrel beam is a W18×50. The architect has set the slab edge at 12 in. from the centerline of the spandrel beam; thus, l_{eos} = 12 in. The deck height, h_d = 2 in. The total slab height, h_s = 6 in.
Solution:
The spandrel beam flange width, b_f= 7.50 in. The slab overhang is,

$$\begin{aligned} l_{oh} &= l_{eos} - \frac{b_f}{2} \\ &= 12 \text{ in.} - \frac{7.50 \text{ in.}}{2} \\ &= 8.25 \text{ in.} \end{aligned}$$

Using Table 5-1, with an 8¼-in. overhang and a 6-in. slab depth, a 10-gage pour-stop thickness is required.

Use a 10-gage pour stop thickness.

Example 5.2—Concrete Slab Overhanging a Spandrel Beam and Reinforced to Support a CMU Wall

For the concrete slab supporting a concrete masonry unit (CMU) wall and overhanging a spandrel beam as illustrated in Figures 5-16 (deck perpendicular) and 5-17 (deck parallel), determine the adequacy of the slab and design the required reinforcement for the following conditions:

Condition A. Deck perpendicular to the spandrel beam as shown in Figure 5-16.

Condition B. Deck parallel to the spandrel beam as shown in Figure 5-17.

Given:
For this example, assume that the controlling strength load combination is 1.4D.

The slab edge is formed with a light-gage metal pour stop that does not contribute to the slab overhang strength. The slab is composed of normal-weight concrete (w_c = 150 lb/ft^3; f'_c = 3,000 psi), the CMU wall is 8-in. thick (w_{blk} = 61 psf), and the story height, h = 14 ft. The architect has specified a 1-in. planned reveal under the CMU wall at the end of the slab overhang to provide for an inward or outward tolerance of 1 in. for the face of block, which results in a worst-case reveal of 2 in.

The architect has set the slab edge such that the distance from the slab edge to the tip of the spandrel beam flange, l_{oh} = 10 in. The deck height, h_d = 2 in. The total slab height, h_s = 4½ in.

The reinforcing steel has f_y = 60 ksi and clear cover, c_c = 1 in.

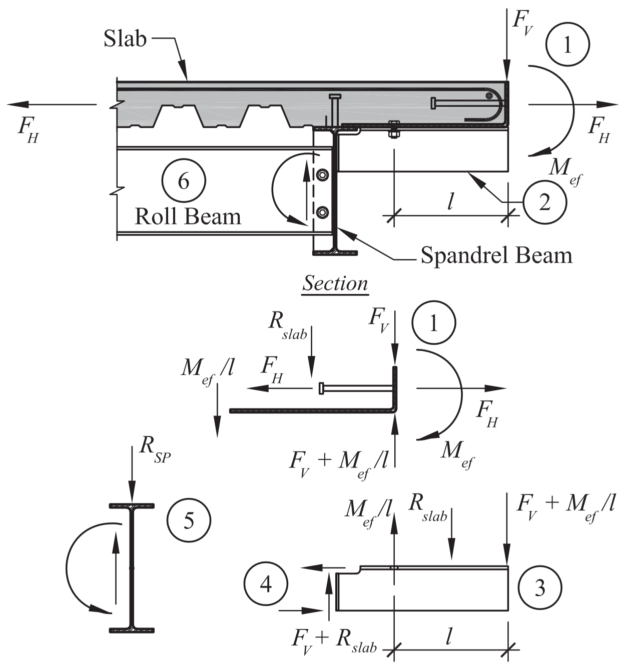
Solution:
Because the slab cantilevers over the spandrel beam, the CMU wall load does not induce torsion in the spandrel beam. The required flexural strength in the slab overhang due to slab self-weight is,

$$\begin{aligned} M_{u\,sw} &= 1.4\left(\frac{w_c h_s l_{oh}^2}{2}\right) \\ &= 1.4\left(\frac{(0.150 \text{ kip/ft}^3)(4\tfrac{1}{2} \text{ in.})(10 \text{ in.})^2}{2(12 \text{ in./ft})^2}\right) \\ &= 0.328 \text{ kip-in./ft} \end{aligned}$$

The weight of the CMU wall (conservatively using the full 14-ft story height) is,

$$\begin{aligned} P_{blk} &= w_{blk} h \\ &= 0.061 \text{ kip/ft}^2 (14 \text{ ft}) \\ &= 0.854 \text{ kip/ft} \end{aligned}$$

Based upon the width of the CMU wall, b_{blk} = 7⅝ in., and the worst-case reveal of 2 in., the moment arm for the wall load



NOTES:

1. The steel edge plate is designed to transfer the vertical facade force and facade moment to the bracket plate. The horizontal façade force is transferred to the slab with a headed stud or deformed bar dowel.

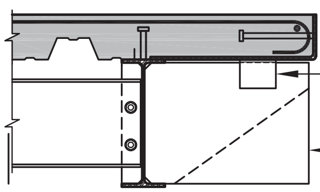
2. The brackets can be made from angles or tees to allow bolting and field adjustments of slab edge.

3. The brackets transfer the façade forces to the spandrel beam.

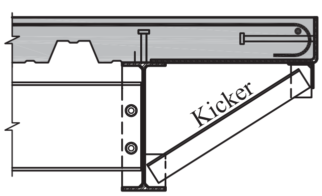
4. The web of the spandrel beam needs to be checked for the out-of-plane forces from the brackets. Full depth stiffeners can be used as brackets if necessary.

5. The spandrel beam must carry torsion between roll beams, kickers or other torsional restraints.

6. Roll beams, kickers or other torsional restraints resist the torsion from the spandrel.



(a) Alternative edge support bracket with full-depth stiffener



(b) Alternative edge support bracket with kicker

Fig. 5-14. Design concept for slab edge detail where slab is neglected except for in-plane forces and brackets are used.

to the tip of the flange of the spandrel beam is,

$$l_{ma} = l_{oh} - \frac{b_{blk}}{2} + 2 \text{ in.}$$
$$= 10 \text{ in.} - \frac{7\frac{5}{8} \text{ in.}}{2} + 2 \text{ in.}$$
$$= 8.19 \text{ in.}$$

The required flexural strength in the slab overhang due to the CMU wall is,

$$M_{u\,blk} = 1.4\, P_{blk} l_{ma}$$
$$= 1.4\,(0.854 \text{ kip/ft})(8.19 \text{ in.})$$
$$= 9.79 \text{ kip-in./ft}$$

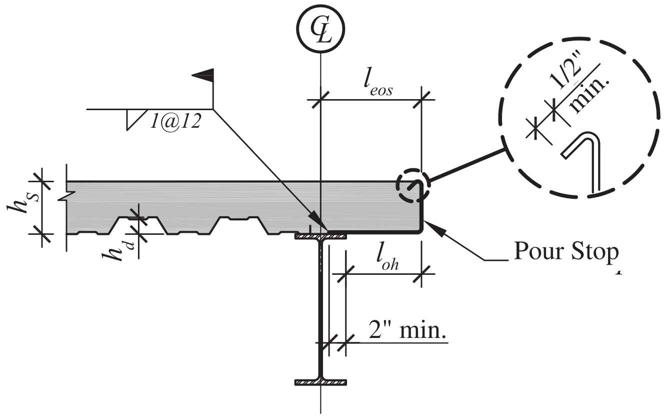


Fig. 5-15. Slab edge with light-gage metal pour stop.

The total required flexural strength is,

$$M_u = M_{u\,sw} + M_{u\,blk}$$
$$= 0.328 \text{ kip-in./ft} + 9.79 \text{ kip-in./ft}$$
$$= 10.1 \text{ kip-in./ft}$$

For Condition A, the deck is perpendicular to the spandrel beam (see Figure 5-16). Try #3 bars ($A_b = 0.11$ in.^{2}) at 18 in. on center, which provides an area of steel A_s per ft of,

$$A_s = \frac{A_b}{s}$$
$$= \frac{0.11 \text{ in.}^2}{18 \text{ in.}}(12 \text{ in./ft})$$
$$= 0.0733 \text{ in.}^2\text{/ft}$$

The effective slab width, $b = 6$ in./ft, which is half the full width because the deck is perpendicular to beam. The distance from the extreme compression fiber to the centroid of the tension reinforcement is,

$$d = h_s - c_c - \frac{d_b}{2}$$
$$= 4\tfrac{1}{2} \text{ in.} - 1 \text{ in.} - \frac{\frac{3}{8} \text{ in.}}{2}$$
$$= 3.31 \text{ in.}$$

By strain compatibility and equilibrium calculations not shown here, the tensile strain in the steel, $\varepsilon_s = 0.0264$. With $\varepsilon_s > \varepsilon_y$ the steel stress can be taken equal to f_y, and the concrete compression block depth is,

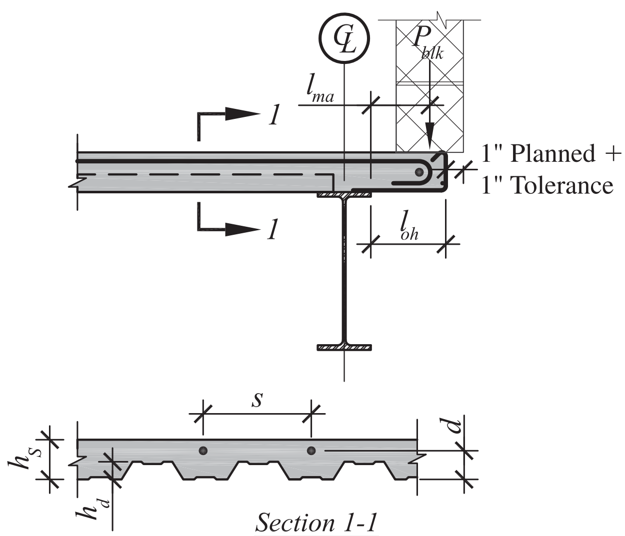


Fig. 5-16. Spandrel beam detail when deck is perpendicular.

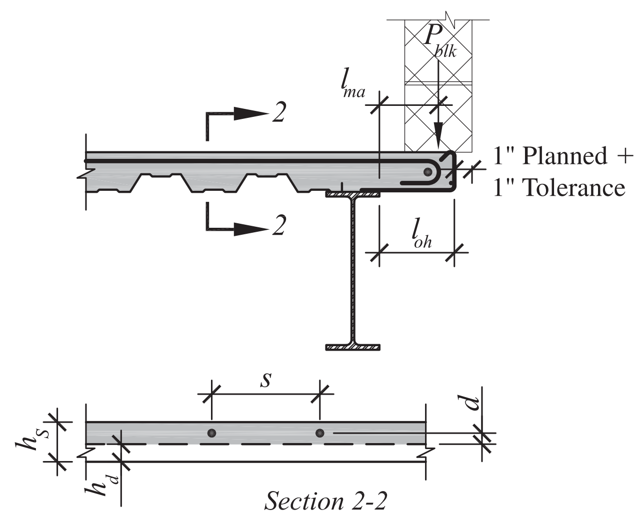


Fig. 5-17. Spandrel beam detail when deck is parallel.

$$a = \frac{A_s f_y}{0.85 f_c' b}$$
$$= \frac{(0.0733 \text{ in.}^2/\text{ft})(60 \text{ ksi})}{0.85(3 \text{ ksi})(6 \text{ in./ft})}$$
$$= 0.287 \text{ in.}$$

Thus,

$$\phi = 0.90 \text{ (since } \varepsilon_s > 0.005\text{)}$$
$$\phi M_n = \phi A_s f_y \left(d - \frac{a}{2}\right)$$
$$= 0.9\left(0.0733 \text{ in.}^2/\text{ft}\right)\left(60 \text{ ksi}\right)\left(3.31 \text{ in.} - \frac{0.287 \text{ in.}}{2}\right)$$
$$= 12.5 \text{ kip-in./ft} \geq M_u \quad \textbf{o.k.}$$

From calculations not shown here, the development length of the hooked ends does not control in this case. For illustration of this check, see Example 5.4. Note that the slab reinforcement also must be fully developed on the other side of the point of maximum moment.

Use #3 reinforcing bars at 18 in. o.c. for Condition A.

Alternatively, Tables 5-2 through 5-9 provide the flexural strength based on concrete compressive strength, deck height, deck orientation, total slab thickness, and amount of reinforcement. The use of these tables will be illustrated in the solution that follows for Condition B.

For Condition B, the deck is parallel to the spandrel beam (see Figure 5-17). Using Table 5-2, #3 reinforcing bars at 8 in. o.c. provide

$$\phi M_n = 10.2 \text{ kip-in.} \geq M_u \quad \textbf{o.k.}$$

Use #3 reinforcing bars at 8 in. o.c. for Condition B.

Comments:
ACI 318-05 utilizes a strain compatibility approach to determine flexural strength. Also, the resistance factor, ϕ, is variable based upon the strain in the tension steel, ε_s. For ε_s less than or equal to ε_y, $\phi = 0.65$. For ε_s greater than or equal to 0.005, $\phi = 0.90$. Linear interpolation is used to determine the value of ϕ between these points.

For brevity, a check of the slab shear strength is not shown in these examples. Designers should verify that the slab has adequate shear strength.

Example 5.3—Bent Plate Pour Stop

For the bent plate pour stop illustrated in Figure 5-18, determine the required thickness based on the maximum slab overhang. Use ASTM A36 material for the bent plate (F_y = 36 ksi).

1. Limit the vertical deflection at the tip of the bent plate pour stop under the wet weight of the concrete to ⅛ in.

2. Check the strength of the plate using the wet weight of the concrete and a 20-psf construction live load.

3. Check the strength of the plate using the wet weight of the concrete and a 250-lb point load at the edge of the pour stop.

4. Determine the required weld between the pour stop and the beam flange.

Given:
For this example, use a load factor of 1.6 for the wet concrete weight, bent-plate self weight, and construction live loads, and assume that these are the only loads applied to the bent plate (the slab reinforcing will be designed for all superimposed loads).

The slab is composed of normal-weight concrete (w_c = 150 lb/ft^3) and the spandrel beam is a W18×50. The architect has set the slab edge at 18 in. from the centerline of the spandrel beam; thus, l_{eos} = 18 in. The total slab height, h_s = 7 in. The overlap of the plate to the weld on the beam flange, s_{weld} = 2 in.

Solution:
The spandrel beam flange width, b_f = 7.50 in. The slab overhang is,

$$l_{oh} = l_{eos} - \frac{b_f}{2}$$
$$= 18 \text{ in.} - \frac{7.50 \text{ in.}}{2}$$
$$= 14.3 \text{ in.}$$

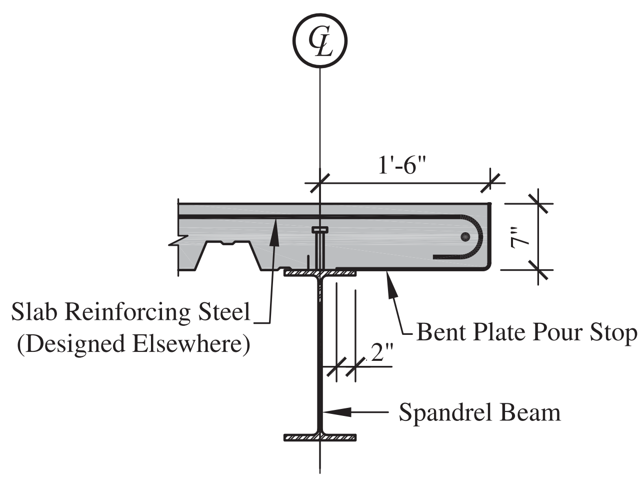


Fig. 5-18. Slab edge with bent plate pour stop.

Try a ¼-in. bent plate thickness. The plate moment of inertia per ft of width (b = 12 in./ft) is,

$$I = \frac{bt_p^3}{12} = \frac{12 \text{ in./ft}(\tfrac{1}{4} \text{ in.})^3}{12} = 0.0156 \text{ in.}^4\text{/ft}$$

The plate plastic section modulus per ft of width is,

$$Z = \frac{bt_p^2}{4} = \frac{12 \text{ in./ft}(\tfrac{1}{4} \text{ in.})^2}{4} = 0.188 \text{ in.}^3\text{/ft}$$

1. Limit the vertical deflection at the tip of the bent plate under the wet weight of the concrete to ⅛ in. The service-load moment at the base of the vertical leg of the bent plate due to lateral fluid pressure from the wet concrete is,

$$M_H = \frac{w_c h_s^3}{6} = \frac{0.150 \text{ kip/ft}^3 (7 \text{ in.})^3}{6(12 \text{ in./ft})^2} = 0.0595 \text{ kip-in./ft}$$

The vertical deflection at the tip due to M_H can be determined assuming that the plate is supported by "pins" at each line of weld (see Figure 5-19), or by approximating the plate as having a fixed support at the tip of the flange (see Figure 5-20). For the former case,

$$\Delta_{M_H} = \frac{M_H}{EI}\left(\frac{l_{oh}s_{weld}}{3} + \frac{l_{oh}^2}{2}\right) = \frac{0.0595 \text{ kip-in./ft}}{29{,}000 \text{ ksi}(0.0156 \text{ in.}^4\text{/ft})} \times \left(\frac{14.3 \text{ in.}(2 \text{ in.})}{3} + \frac{(14.3 \text{ in.})^2}{2}\right) = 0.0147 \text{ in.}$$

For the latter case (and subsequent calculations in this example),

$$\Delta_{M_H} = \frac{M_H l_{oh}^2}{2EI} = \frac{0.0595 \text{ kip-in./ft}(14.3 \text{ in.})^2}{2(29{,}000 \text{ ksi})(0.0156 \text{ in.}^4\text{/ft})} = 0.0134 \text{ in.}$$

The vertical deflection at the tip due to the wet weight of the concrete (from AISC Manual Table 3-23, case 19) is,

$$\Delta_{w_c} = \frac{w_c h_s l_{oh}^4}{8EI} = \frac{0.150 \text{ kip/ft}^3 (7 \text{ in.})(14.3 \text{ in.})^4}{8(29{,}000 \text{ ksi})(0.0156 \text{ in.}^4\text{/ft})(12 \text{ in./ft})^2} = 0.0842 \text{ in.}$$

The vertical deflection at the tip due to the self-weight of the bent plate (from a combination of AISC *Manual* Table 3-23, cases 19 and 22) is,

$$\Delta_{w_s} = \frac{w_s t_p l_{oh}^4}{8EI} + \frac{w_s t_p h_s l_{oh}^3}{3EI} = \frac{0.490 \text{ kip/ft}^3 (\tfrac{1}{4} \text{ in.})(14.3 \text{ in.})^4}{8(29{,}000 \text{ ksi})(0.0156 \text{ in.}^4\text{/ft})(12 \text{ in./ft})^2} + \frac{0.490 \text{ kip/ft}^3 (\tfrac{1}{4} \text{ in.})(7 \text{ in.})(14.3 \text{ in.})^3}{3(29{,}000 \text{ ksi})(0.0156 \text{ in.}^4\text{/ft})(12 \text{ in./ft})^2} = 0.0227 \text{ in.}$$

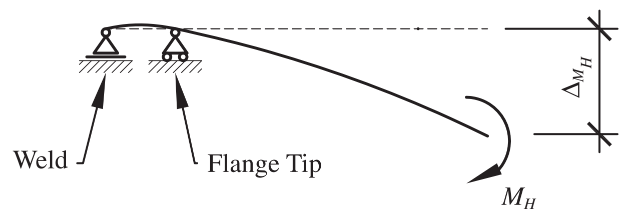


Fig. 5-19. Analysis model with pinned supports.

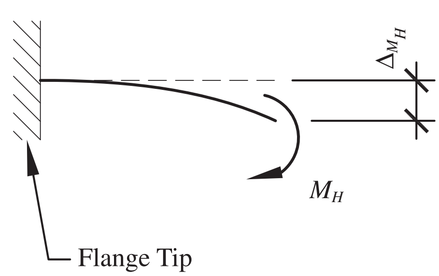


Fig. 5-20. Analysis model with fixed support.

The total vertical deflection is,

$$\Delta_{tot} = \Delta_{M_H} + \Delta_{w_c} + \Delta_{w_s}$$
$$= 0.0134 \text{ in.} + 0.0842 \text{ in.} + 0.0227 \text{ in.}$$
$$= 0.120 \text{ in.} < \tfrac{1}{8} \text{ in.} \quad \textbf{o.k.}$$

2. Check the strength of the bent plate using the wet weight of the concrete and a 20-psf construction live load. The required flexural strength due to lateral fluid pressure from the wet concrete is,

$$M_{u\,H} = 1.6M_H$$
$$= 1.6(0.0595 \text{ kip-in./ft})$$
$$= 0.0952 \text{ kip-in./ft}$$

The required flexural strength due to the wet weight of the concrete is,

$$M_{u\,w_c} = 1.6\left(\frac{w_c h_s l_{oh}^2}{2}\right)$$
$$= 1.6\left(\frac{(0.150 \text{ kip/ft}^3)(7 \text{ in.})(14.3 \text{ in.})^2}{2(12 \text{ in./ft})^2}\right)$$
$$= 1.19 \text{ kip-in./ft}$$

The required flexural strength due to the uniform construction live load is,

$$M_{u\,cll} = 1.6\left(\frac{w_{cll} l_{oh}^2}{2}\right)$$
$$= 1.6\left(\frac{0.020 \text{ kip/ft}^2\,(14.3 \text{ in.})^2}{2(12 \text{ in./ft})}\right)$$
$$= 0.273 \text{ kip-in./ft}$$

The total required flexural strength is,

$$M_u = M_{u\,H} + M_{u\,wc} + M_{ucll}$$
$$= 0.0952 \text{ kip-in./ft} + 1.19 \text{ kip-in./ft}$$
$$+ 0.273 \text{ kip-in./ft}$$
$$= 1.56 \text{ kip-in./ft}$$

From AISC Specification Section F11, the available flexural strength is,

$$\phi_b M_n = \phi_b F_y Z$$
$$= 0.90(36 \text{ ksi})(0.188 \text{ in.}^3/\text{ft})$$
$$= 6.09 \text{ kip-in./ft} \ge M_u \quad \textbf{o.k.}$$

The $1.6F_yS$ limit need not be checked because $Z/S = 1.5$ for a rectangular plate in bending.

3. Check the strength of the bent plate using the wet weight of the concrete and a 250-lb point load. The effective width, b_{eff}, of the pour stop that resists the effects of the 250-lb point load is determined assuming that the load spreads at a 45° angle in both directions. Thus,

$$b_{eff} = 2\tan(45°)\, l_{oh}$$
$$= 2\tan(45°)(14.3 \text{ in.})$$
$$= 28.6 \text{ in.}$$

This approach is conservative because it neglects any increase in effective width due to the stiffening effect of the vertical leg of the bent plate, which could be substantial. Alternatively, the designer can choose to consider the stiffening effect of the vertical leg.

The required flexural strength due to the uniform construction live load is,

$$M_{u\,P_c} = 1.6P_c l_{oh}\left(\frac{12 \text{ in./ft}}{b_{eff}}\right)$$
$$= 1.6(0.250 \text{ kip})(14.3 \text{ in.})\left(\frac{12 \text{ in./ft}}{28.6 \text{ in.}}\right)$$
$$= 2.40 \text{ kip-in./ft}$$

The total required flexural strength is,

$$M_u = M_{u\,H} + M_{u\,w_c} + M_{u\,P_c}$$
$$= 0.0952 \text{ kip-in./ft} + 1.19 \text{ kip-in./ft}$$
$$+ 2.40 \text{ kip-in./ft}$$
$$= 3.69 \text{ kip-in./ft}$$

As calculated previously, the available flexural strength is,

$$\phi_b M_n = 6.09 \text{ kip-in./ft} \ge M_u \quad \textbf{o.k.}$$

Use a ¼-in. bent plate pour stop thickness.

4. Determine the required weld between the bent plate pour stop and the spandrel beam flange. Try a 3⁄16-in. fillet weld with $F_{EXX} = 70$ ksi. The basic weld strength per AISC Manual page 8-8 is,

$$\phi R_n = 1.392Dl$$

where

D = weld size in sixteenths of an inch
l = length, in.

Note that the loading angle is 90° and a 1.5 weld strength increase factor is also applicable.

The required length of fillet weld per ft of width (see Figure 5-21) using the larger value of M_u from Parts 2 and 3 of this Example is,

$$
\begin{aligned}
l_{min} &= \frac{M_u}{1.392 \text{ kip/in.}(1.5)D\, s_{weld}} \\
&= \frac{3.69 \text{ kip-in./ft}}{1.392 \text{ kip/in.}(1.5)(3)(2 \text{ in.})} \\
&= 0.295 \text{ in./ft}
\end{aligned}
$$

The minimum permissible fillet weld length is 1½ in. per Specification Section J2.2b.

Use intermittent 3⁄16-in. fillet welds, 1½-in. long at 12 in. on center.

Comments:
The ⅛-in. deflection limit used in this example is arbitrary. SDI uses ¼ in. for the deflection criterion of light-gage metal pour stops. The designer should determine appropriate deflection limits based on project conditions. For completeness, the deflection due to the self-weight of the bent plate was included here, although for practical purposes this could be ignored.

The bent plate pour stop in this example could have been selected from Table 5-10 (normal-weight concrete), by entering the table with a 7-in. slab thickness and a 14-in. overhang (beyond the edge of the beam flange). The table shows that a ¼-in. plate thickness is required. Similarly, Table 5-11 can be used for lightweight concrete.

Use of the 1.6 load factor for both the construction loads and the wet weight of the concrete reflects the typical practice used to determine tabulated pour stop values in manufacturer's catalogs. The 250-lb construction point load is referenced in Table 1 of SEI/ASCE 37 (ASCE, 2002).

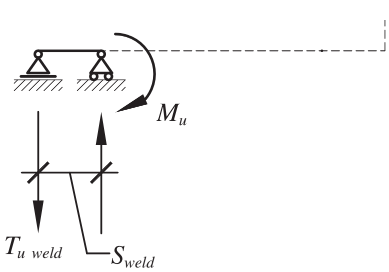


Fig. 5-21. Analysis model for weld design.

Example 5.4—Bent Plate Pour Stop with Headed Studs Engaging the Slab Reinforcement to Support a Façade

For the bent plate pour stop illustrated in Figure 5-22,

1. Design the headed studs to transfer shear and tension due to the weight of the curtain wall and wind forces.

2. Design the slab flexural reinforcement.

Given:
For this example, assume that the controlling strength load combination is $1.2D + 1.6L + 0.8W$.

The slab is composed of lightweight concrete (w_c = 115 lb/ft^3; f'_c = 4,000 psi), the spandrel beam is a W18×35, and the story height, h = 14 ft. The architect has set the slab edge at 12 in. from the centerline of the spandrel beam; thus, l_{eos} = 12 in. The deck height, h_d = 3 in. The total slab height, h_s = 6¼ in. Two headed studs are used at each curtain wall attachment point with spacing between them, s_s = 12 in. The clear cover between the hooked end of the slab bars and the interior face of the pour stop, c_c = ¾ in. (1 in. to the top of slab)

The superimposed dead load, w_{SIDL} = 15 psf. The floor live load, w_{LL} = 100 psf. Each attachment supports two stories of curtain wall cladding (w_{curt} = 15 psf) attached directly to the ¼-in.-thick bent plate pour stop at vertical mullions with

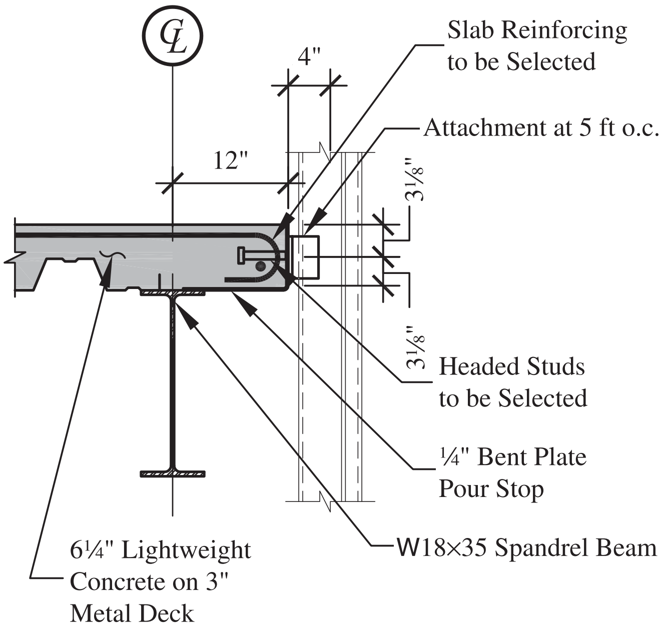


Fig. 5-22. Section of slab edge with bent plate pour stop with headed studs acting as façade anchorage.

spacing, s_c = 5 ft. The eccentricity between the CG of the curtain wall and the slab edge, e_{cw} = 4 in. (see Figure 5-23). The cladding is exposed to a "components and cladding" wind pressure (or suction), p_w = 25 psf. Assume that the curtain wall is braced for out-of-plane loads at each floor level.

Solution:

The spandrel beam flange width, b_f = 6.00 in. The slab overhang is,

$$\begin{aligned} l_{oh} &= l_{eos} - \frac{b_f}{2} \\ &= 12 \text{ in.} - \frac{6.00 \text{ in.}}{2} \\ &= 9.00 \text{ in.} \end{aligned}$$

1. Design the headed studs to transfer the shear and tension forces. The height of cladding suspended from the spandrel beam is,

$$\begin{aligned} h_c &= 2h \\ &= 2(14 \text{ ft}) \\ &= 28 \text{ ft} \end{aligned}$$

The required strength in tension due to the wind load is,

$$\begin{aligned} N_{u\,w} &= 0.8F_H \\ &= 0.8p_w h s_c \\ &= 0.8\left(0.025 \text{ kip/ft}^2\right)(14 \text{ ft})(5 \text{ ft}) \\ &= 1.40 \text{ kips} \end{aligned}$$

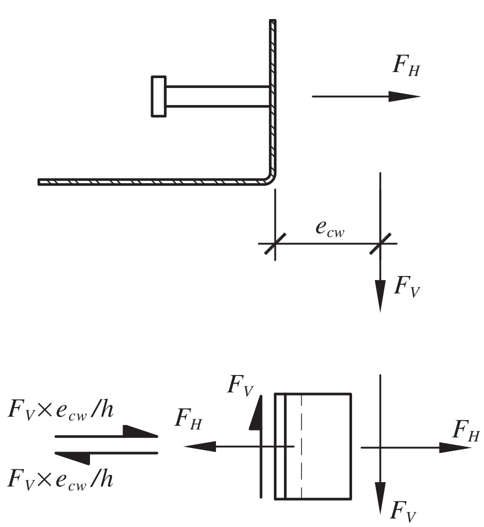


Fig. 5-23. Forces acting on attachments.

The required strength in shear due to the curtain wall load is,

$$\begin{aligned} V_{u\,cw} &= 1.2F_V \\ &= 1.2w_{curt}h_c s_c \\ &= 1.2\left(0.015 \text{ kip/ft}^2\right)(28 \text{ ft})(5 \text{ ft}) \\ &= 2.52 \text{ kips} \end{aligned}$$

Note that the eccentricity of the curtain wall with respect to the slab edge also creates a horizontal force on the slab, which is,

$$\begin{aligned} N_{u\,cw} &= 1.2F_V \frac{e_{cw}}{h} \\ &= V_{u\,cw} \frac{e_{cw}}{h} \\ &= 2.52 \text{ kips} \left(\frac{4 \text{ in.}}{14 \text{ ft}(12 \text{ in./ft})}\right) \\ &= 0.0600 \text{ kip} \end{aligned}$$

This force is usually balanced at each intermediate floor by a similar force in compression from the adjacent panel. Even if it were not, the horizontal force in this case is small and has been neglected in subsequent calculations.

With two headed studs at each façade attachment point, $n_s = 2$, and the required tensile and shear strengths per stud are,

$$\begin{aligned} N_u &= \frac{N_{u\,w}}{n_s} \\ &= \frac{1.40 \text{ kips}}{2} \\ &= 0.700 \text{ kip} \end{aligned}$$

$$\begin{aligned} V_u &= \frac{V_{u\,cw}}{n_s} \\ &= \frac{2.52 \text{ kips}}{2} \\ &= 1.26 \text{ kips} \end{aligned}$$

From Tables 5-14 and 5-15, the design tensile and shear strengths for ¾-in.-diameter headed studs with a 6-in. embedment and a 6¼-in.-thick lightweight concrete slab are, ϕN_n = 3.27 kips and ϕV_n = 1.93 kips, respectively. Group effects can be ignored because the studs spacing, s_s = 12 in., exceeds three times the edge distance (3⅛ in.).

$$\begin{aligned} \frac{N_u}{\phi N_n} &= \frac{0.700 \text{ kips}}{3.27 \text{ kips}} \\ &= 0.214 \end{aligned}$$

$$\frac{V_u}{\phi V_n} = \frac{1.26 \text{ kips}}{1.93 \text{ kips}}$$
$$= 0.653$$

Because these individual ratios are both greater than 0.2, the interaction of tension and shear must be investigated following ACI 318-05 Equation D-31.

$$\frac{V_u}{\phi V_n} + \frac{N_u}{\phi N_n} \le 1.2$$
$$0.653 + 0.214 \le 1.2$$
$$0.867 \le 1.2 \quad \textbf{o.k.}$$

Use two ¾-in. headed studs.

2. Design the slab flexural reinforcement. Because the deck is parallel to the spandrel beam, the critical section for moment in the slab is at the first flute in the backspan (see Figure 5-24). If this location is known, the reduced moment can be used. Otherwise, the moment at the centerline of the spandrel beam can be used.

With the self-weight of the slab resisted by the bent-plate pour stop, only superimposed loads are considered in the design of the slab flexural reinforcement. For the superimposed dead load and floor live load,

$$w_u = 1.2w_{SIDL} + 1.6w_{LL}$$
$$= 1.2\left(0.015 \text{ kip/ft}^2\right) + 1.6\left(0.100 \text{ kip/ft}^2\right)$$
$$= 0.178 \text{ kip/ft}^2$$

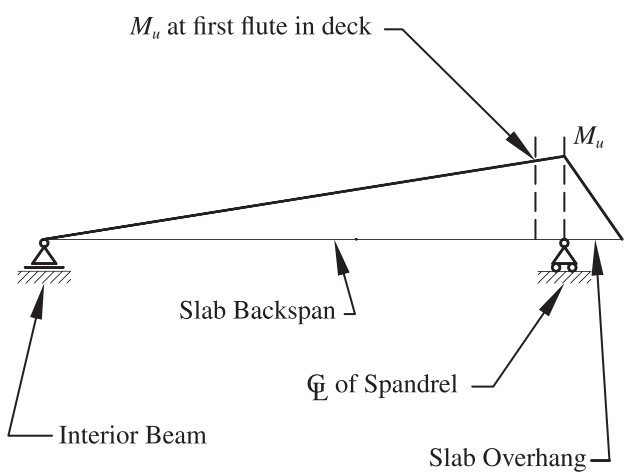


Fig. 5-24. Moment diagram for slab.

The effective width, b_{eff}, of the slab that resists the vertical load from the curtain wall, $V_{u\,cw}$, is determined assuming that the load spreads at a 45° angle in both directions. Thus,

$$b_{eff} = s_s + 2\tan(45°)\,l_{oh}$$
$$= 12 \text{ in.} + 2\tan(45°)(9.00 \text{ in.})$$
$$= 30.0 \text{ in.}$$

The force from this load when adjusted to a 12-in. width is,

$$P_u = V_{u\,cw}\left(\frac{12 \text{ in./ft}}{b_{eff}}\right)$$
$$= 2.52 \text{ kips}\left(\frac{12 \text{ in./ft}}{30.0 \text{ in.}}\right)$$
$$= 1.01 \text{ kip/ft}$$

The total required flexural strength (ignoring the slight reduction in moment from the spandrel beam centerline to the location of the critical section) is,

$$M_u = \frac{w_u l_{eos}^2}{2} + P_u l_{eos}$$
$$= \frac{0.178 \text{ kips / ft}^2\,(12 \text{ in.})^2}{2(12 \text{ in./ft})} + 1.01 \text{ kip/ft}(12 \text{ in.})$$
$$= 13.2 \text{ kip-in./ft}$$

Try #4 reinforcing bars at 16 in. on center. From Table 5-5, ϕM_n = 15.3 kip-in./ft and A_s = 0.150 in.^{2}/ft, but the development of the hooked ends must also be checked.

As shown in Figure 5-25, the length of the hooked bar beyond the critical section of slab depends on the location of the critical section (the first deck flute). The actual location of this flute is not always known, but it is reasonable to assume that it cannot be any closer to the slab edge than the edge of the ¾-in.-diameter composite shear stud connector shown in Figure 5-25. With the stud at the centerline of the spandrel beam,

$$l_h = l_{eos} - c_c - \frac{d_b}{2} + \frac{d_{stud}}{2}$$
$$= 12 \text{ in.} - ¾ \text{ in.} - \frac{½ \text{ in.}}{2} + \frac{¾ \text{ in.}}{2}$$
$$= 11.4 \text{ in.}$$

When the actual location of the first flute is known, a more accurate calculation can be performed.

The required development length can be determined using ACI 318 Section 12.5. For illustration purposes, assume l_{dh} = 12 in., which exceeds l_h. Therefore, A_s must be reduced to account for partial development of the bar. Additionally, the

slab steel must resist out-of-plane tension on the slab edge caused by wind loads and the eccentricity of the curtain wall. The effective area of reinforcing steel in the slab (reduced to account for out-of-plane tension on the slab edge, $2N_u$) is,

$$A_{se} = \frac{l_h}{l_{dh}}\left(A_s - \frac{2N_u}{\phi f_y}\frac{(12\text{ in./ft})}{(b_{eff})}\right)$$

$$= \frac{11.4\text{ in.}}{12\text{ in.}}\left(0.150\text{ in.}^2\text{/ft} - \frac{2(0.700\text{ kip})}{0.90(60\text{ ksi})}\frac{(12\text{ in./ft})}{(30.0\text{ in.})}\right)$$

$$= 0.133\text{ in.}^2\text{/ft}$$

Note that $2N_u$ is used here because two headed studs are located at each attachment point.

The effective slab width, $b = 12$ in./ft, which is the full width because the deck is parallel to the beam. The distance from the extreme compression fiber to the centroid of the tension reinforcement is,

$$d = h_s - h_d - c_c - \frac{d_b}{2}$$

$$= 6\tfrac{1}{4}\text{ in.} - 3\text{ in.} - \tfrac{3}{4}\text{ in.} - \frac{\tfrac{1}{2}\text{ in.}}{2}$$

$$= 2\tfrac{1}{4}\text{ in}$$

By strain compatibility and equilibrium calculations not shown here, the tensile strain in the steel, $\varepsilon_s = 0.0231$. With $\varepsilon_s > \varepsilon_y$ the steel stress can be taken equal to f_y, and the concrete compression block depth is,

$$a = \frac{A_{se} f_y}{0.85 f_c' b}$$

$$= \frac{0.133\text{ in.}^2\text{/ft }(60\text{ ksi})}{0.85(4\text{ ksi})(12\text{ in./ft})}$$

$$= 0.196\text{ in./ft}$$

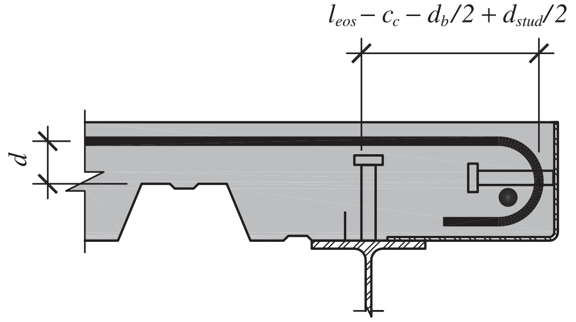


Fig. 5-25. Slab dimensions for analysis.

Thus,

$$\phi = 0.90 \text{ (since } \varepsilon_s > 0.005)$$

$$\phi M_n = \phi A_{se} f_y\left(d - \frac{a}{2}\right)$$

$$= 0.90(0.133\text{ in.}^2)(60\text{ ksi})\left(2\tfrac{1}{4}\text{ in.} - \frac{0.196\text{ in.}}{2}\right)$$

$$= 15.5\text{ kip-in./ft} \geq M_u \quad \textbf{o.k.}$$

Note that the slab reinforcement also must be fully developed on the other side of the point of maximum moment.

Use #4 reinforcing bars at 16 in. o.c.

Note that this bar spacing violates the maximum bar spacing for beams and slabs presented in ACI 318-05 Section 10.6.4. The ACI criterion is intended to limit surface cracks in concrete. In this example, it is assumed that the slab will be covered with a finished floor; therefore surface cracks are not a concern, reinforcing steel may be added in the form of additional bars that supplement the typical reinforcing steel. It is important, however, that enough of the steel be developed at the critical section.

Comments:
The bent plate pour stop thickness in this example could have been selected from Table 5-11 as 3⁄16 in. In this application, however, a ¼-in.-thick pour stop is used to facilitate welding the studs to the plate. Generally, curtain wall and metal panel cladding loads can be transferred to slabs with headed studs. This approach usually is not adequate to support heavier façades, such as precast-concrete panels, particularly where the deck flutes are parallel to the spandrel beam.

The reinforcing steel along the slab edges may be in the form of additional bars that supplement the typical slab reinforcing steel. Although its effect has been neglected in this example, the amount of flexural reinforcement along the slab edges can be minimized by considering the flexural strength of the flange of the spandrel beam.

Note that some of the out-of-plane wind load will be resisted through the bent plate pour stop, though this has been neglected in this example.

For brevity, a check of the slab shear strength is not shown. Designers should verify that the slab has adequate shear strength.

Example 5.5—Bent Plate Pour Stop Supporting a Façade with the Slab Ignored

For the bent plate pour stop illustrated in Figure 5-26, determine the required plate thickness and verify that the vertical deflection of the bent plate does not exceed 3⁄16 in. for the following conditions:

Condition A. The vertical load, F_V = 1.8 kips, is applied at the face of the bent plate (eccentricity, e_f = 5 in., is resolved in the design of the façade).

Condition B. F_V is applied at e_f from the face of the bent plate (eccentricity is not resolved in the design of the façade).

Condition C. F_V is applied at e_f from the face of the bent plate and the horizontal load, F_H = 1.8 kips, is applied at eccentricity, e_v = 3½ in., above the bottom of the bent plate. Also determine the required weld size between the plate and the beam flange.

In all cases, design the bent plate to resist all of the façade loads without relying on the concrete slab flexural strength. Use ASTM A36 material for the bent plate (F_y = 36 ksi).

Given:

For this example, assume that the controlling strength load combination is 1.4D (D for deflection) for Conditions A and B, and 1.2D + 1.6W (D + W for deflection) for Condition C. F_V is the cladding dead load and F_H is the wind load. The bent plate pour stop is adequate to resist the wet weight of concrete. The spandrel beam is adequate to resist the torsional loads from the bent plate. Once cured, the concrete slab is adequate to support its own weight and superimposed floor loads.

The architect has set the slab edge such that the distance from the slab edge to the tip of the spandrel beam flange, l_{oh} = 9 in. The total slab height, h_s = 7½ in. Thus, the length of the vertical leg of the bent plate, l_{vert} = 7½ in. The overlap of the bent plate to the weld on the beam flange, s_{weld} = 2½ in.

Solution:

For Condition A, there is no moment in the vertical leg of the bent plate, but moment is developed in the horizontal leg (see Figure 5-27).

Try a 7⁄16-in. bent plate thickness. The effective width, b_{eff}, of the bent plate that resists the effects of F_V is determined assuming that the load spreads at a 45° angle in both directions (see Figure 5-28). Thus,

$$\begin{aligned} b_{eff} &= 2\tan(45°)\, l_{oh} \\ &= 2\tan(45°)(9 \text{ in.}) \\ &= 18.0 \text{ in.} \end{aligned}$$

The plate moment of inertia is,

$$\begin{aligned} I &= \frac{b_{eff}\, t_p^3}{12} \\ &= \frac{18.0 \text{ in.}\,(7/16 \text{ in.})^3}{12} \\ &= 0.126 \text{ in.}^4 \end{aligned}$$

The plate plastic section modulus is,

$$\begin{aligned} Z &= \frac{b_{eff}\, t_p^2}{4} \\ &= \frac{18.0 \text{ in.}\,(7/16 \text{ in.})^2}{4} \\ &= 0.861 \text{ in.}^3 \end{aligned}$$

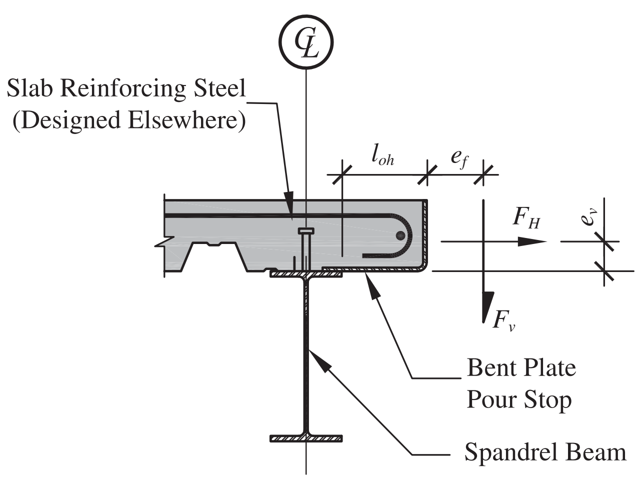


Fig. 5-26. Slab edge with bent plate as pour stop supporting façade loads.

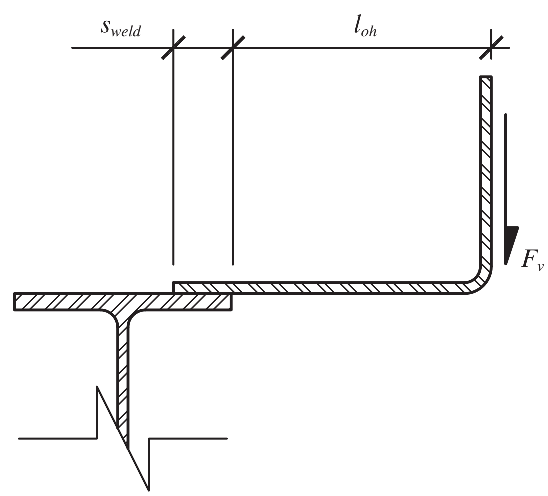


Fig. 5-27. Loading for Condition A.

For the strength load combination 1.4D, the required flexural strength is,

$$\begin{aligned} M_u &= 1.4 F_V l_{oh} \\ &= 1.4(1.8 \text{ kips})(9 \text{ in.}) \\ &= 22.7 \text{ kip-in.} \end{aligned}$$

From AISC *Specification* Section F11, the available flexural strength is,

$$\begin{aligned} \phi_b M_n &= \phi_b F_y Z \\ &= 0.90(36 \text{ ksi})(0.861 \text{ in.}^3) \\ &= 27.9 \text{ kip-in.} \geq M_u \quad \textbf{o.k.} \end{aligned}$$

The $1.6F_yS$ limit need not be checked because $Z/S = 1.5$ for a rectangular plate in bending.

For deflection, using AISC *Manual* Table 3-23, case 26,

$$\begin{aligned} \Delta_V &= \frac{F_V l_{oh}^2}{3EI}(l_{oh} + s_{weld}) \\ &= \frac{1.8 \text{ kips }(9 \text{ in.})^2}{3(29{,}000 \text{ ksi})(0.126 \text{ in.}^4)}(9 \text{ in.} + 2\tfrac{1}{2} \text{ in.}) \\ &= 0.153 \text{ in.} \leq \tfrac{3}{16} \text{ in.} \quad \textbf{o.k.} \end{aligned}$$

Use a 7⁄16-in. bent plate thickness for Condition A.

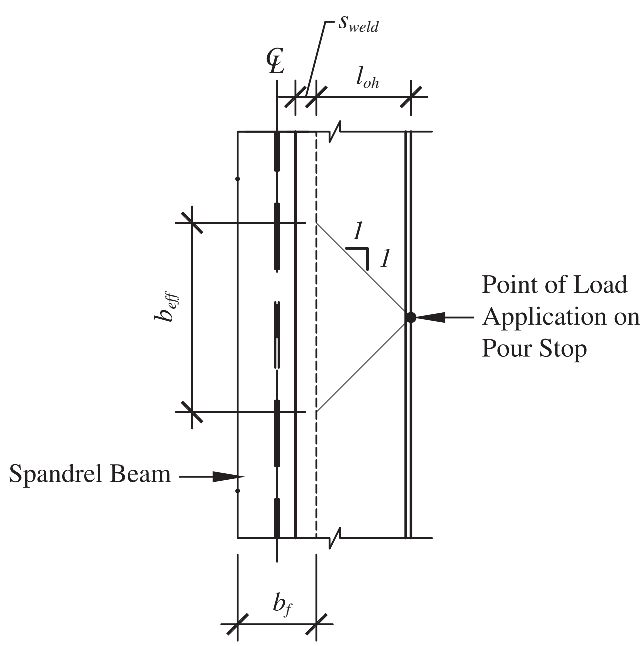


Fig. 5-28. Plan detail.

For Condition B, the loading is the same as for Condition A but with an additional moment in the bent plate due to the cladding eccentricity (see Figures 5-29 and 5-30).

Try a ½-in. bent plate thickness. As calculated previously, $b_{eff} = 18.0$ in. The plate moment of inertia is,

$$\begin{aligned} I &= \frac{b_{eff} t_p^3}{12} \\ &= \frac{18.0 \text{ in. } (\tfrac{1}{2} \text{ in.})^3}{12} \\ &= 0.188 \text{ in.}^4 \end{aligned}$$

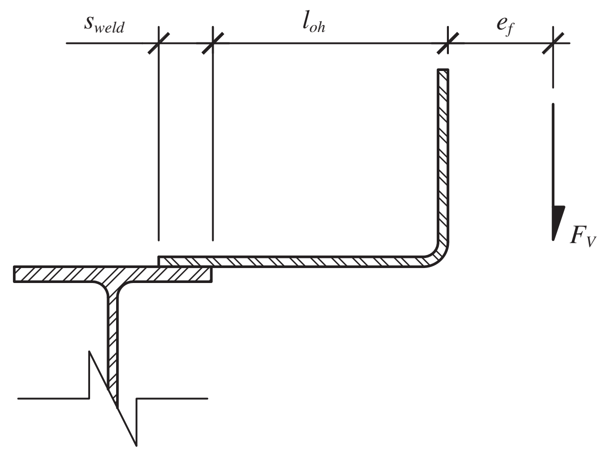


Fig. 5-29. Loading for Condition B.

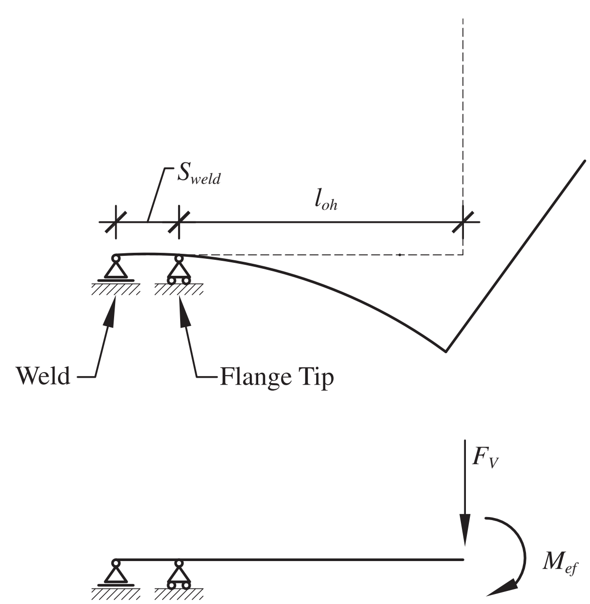


Fig. 5-30. Analysis model for Condition B.

The plate plastic section modulus is,

$$Z = \frac{b_{eff} t_p^2}{4}$$
$$= \frac{18.0 \text{ in.} \left(\tfrac{1}{2} \text{ in.}\right)^2}{4}$$
$$= 1.13 \text{ in.}^3$$

For the strength load combination 1.4*D*, the required flexural strength is,

$$M_u = 1.4 F_V \left(l_{oh} + e_f\right)$$
$$= 1.4\left(1.8 \text{ kips}\right)(9 \text{ in.} + 5 \text{ in.})$$
$$= 35.3 \text{ kip-in.}$$

From AISC *Specification* Section F11, the available flexural strength is,

$$\phi_b M_n = \phi_b F_y Z$$
$$= 0.90\left(36 \text{ ksi}\right)\left(1.13 \text{ in}^3\right)$$
$$= 36.6 \text{ kip-in.} \geq M_u \quad \textbf{o.k.}$$

For deflection, using AISC *Manual* Table 3-23, case 26, and an additional term to account for the effect of the eccentricity,

$$\Delta_V = \frac{F_V l_{oh}^2}{3EI}\left(l_{oh} + s_{weld}\right) + \frac{F_V e_f}{EI}\left(\frac{l_{oh} s_{weld}}{3} + \frac{l_{oh}^2}{2}\right)$$
$$= \frac{1.8 \text{ kips } (9 \text{ in.})^2}{3(29{,}000 \text{ ksi})(0.188 \text{ in.}^4)}\left(9 \text{ in.} + 2\tfrac{1}{2} \text{ in.}\right)$$
$$+ \frac{1.8 \text{ kips}(5 \text{ in.})}{(29{,}000 \text{ ksi})(0.188 \text{ in.}^4)}$$
$$\times\left(\frac{9 \text{ in.}\left(2\tfrac{1}{2} \text{in.}\right)}{3} + \frac{(9 \text{ in.})^2}{2}\right)$$
$$= 0.182 \text{ in.} \leq \tfrac{3}{16} \text{ in.} \quad \textbf{o.k.}$$

Use a ½-in. bent plate thickness for Condition B.

For Condition C, the loading is similar to Condition B but with an additional moment in the vertical and horizontal legs of the bent plate due to the horizontal load F_H is applied at a distance e_v above the bottom of the plate (see Figures 5-31 and 5-32). Also, a different load combination is used.

Try a ⅝-in. bent plate thickness. As calculated previously, b_{eff} = 18.0 in. The plate moment of inertia is,

$$I = \frac{b_{eff} t_p^3}{12}$$
$$= \frac{18.0 \text{ in. } (\tfrac{5}{8} \text{ in.})^3}{12}$$
$$= 0.366 \text{ in.}^4$$

The plate plastic section modulus is,

$$Z = \frac{b_{eff} t_p^2}{4}$$
$$= \frac{18.0 \text{ in. } (\tfrac{5}{8} \text{ in.})^2}{4}$$
$$= 1.76 \text{ in.}^3$$

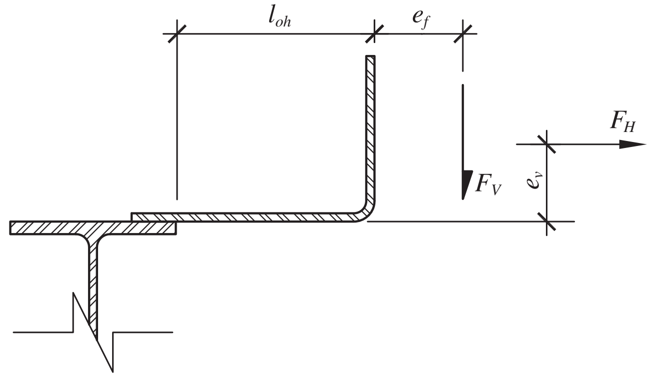


Fig. 5-31. Loading for Condition C.

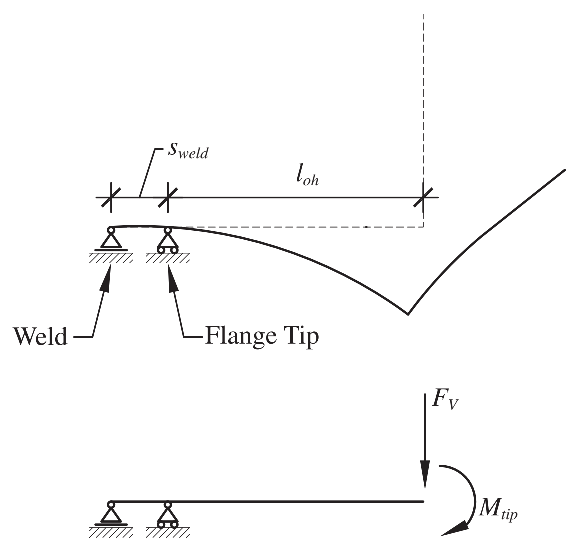


Fig. 5-32. Analysis model for Condition C.

For the strength load combination $1.2D + 1.6W$, the required flexural strength is,

$$\begin{aligned} M_u &= 1.2F_V\left(l_{oh} + e_f\right) + 1.6F_H e_v \\ &= 1.2(1.8 \text{ kips})(9 \text{ in.} + 5 \text{ in.}) \\ &\quad + 1.6(1.8 \text{ kips})(3\tfrac{1}{2} \text{ in.}) \\ &= 40.3 \text{ kip-in.} \end{aligned}$$

From AISC *Specification* Section F11, the available flexural strength is,

$$\begin{aligned} \phi_b M_n &= \phi_b F_y Z \\ &= 0.90(36 \text{ ksi})\left(1.76 \text{ in}^3\right) \\ &= 57.0 \text{ kip-in.} \geq M_u \quad \textbf{o.k.} \end{aligned}$$

For deflection, using AISC *Manual* Table 3-23, case 26, and an additional term to account for the effect of the eccentricities,

$$\begin{aligned} \Delta_V &= \frac{F_V l_{oh}^2}{3EI}\left(l_{oh} + s_{weld}\right) \\ &\quad + \frac{F_V e_f + F_H e_v}{EI}\left(\frac{l_{oh} s_{weld}}{3} + \frac{l_{oh}^2}{2}\right) \\ &= \frac{1.8 \text{ kips } (9 \text{ in.})^2}{3(29{,}000 \text{ ksi})(0.366 \text{ in.}^4)}\left(9 \text{ in.} + 2\tfrac{1}{2} \text{ in.}\right) \\ &\quad + \frac{1.8 \text{ kips } (5 \text{ in.}) + 1.8 \text{ kips } (3\tfrac{1}{2} \text{ in.})}{(29{,}000 \text{ ksi})(0.366 \text{ in.}^4)} \\ &\quad \times \left(\frac{9 \text{ in.}\left(2\tfrac{1}{2} \text{ in.}\right)}{3} + \frac{(9 \text{ in.})^2}{2}\right) \\ &= 0.122 \text{ in.} \leq \tfrac{3}{16} \text{ in.} \quad \textbf{o.k.} \end{aligned}$$

Use a ⅝-in. bent plate thickness for Condition C.

Try a ¼-in. fillet weld with F_{EXX} = 70 ksi between the bent plate pour stop and the spandrel beam flange. As developed previously in Example 5.3, the required length of fillet weld per ft of width is,

$$\begin{aligned} l_{min} &= \frac{M_u}{1.392 \text{ kips/in.}(1.5)D\ s_{weld}} \\ &= \frac{40.3 \text{ kip-in.}}{1.392 \text{ kips/in.}(1.5)(4)\left(2\tfrac{1}{2} \text{ in.}\right)} \\ &= 1.93 \text{ in.} \end{aligned}$$

The minimum permissible fillet weld length is 1½ in. per *Specification* Section J2.2b.

Use intermittent ¼-in. fillet welds, 2-in. long at 12 in. on center.

Comments:
Even for this relatively small overhang, the steel plate becomes thick when it must support cladding load and out-of-plane wind loads. There may be more cost effective alternatives to using a thick plate, including the use of headed studs or welded bar couplers and threaded reinforcing bars to transfer load into the slab. One such alternative is illustrated in Example 5.6.

Example 5.6—Bent Plate Pour Stop Supporting a Façade with the Slab Ignored, Except for Welded Bar Couplers Engaging Threaded Reinforcing Bars to Resist Out-of-Plane Forces

For the bent plate pour stop illustrated in Figure 5-33, repeat Example 5.5 Condition C, but with a welded bar coupler attached to the backside of the bent plate and engaging threaded reinforcing bars to transfer all out-of-plane forces into the slab. Because the out-of-plane forces are no longer resisted by the bent plate, use the load combination $1.4D$.

Solution:
The horizontal force, F_H, acting on the vertical leg of the bent plate is transferred directly to the slab by the welded bar coupler and threaded reinforcing bar. A moment, M_{ef},

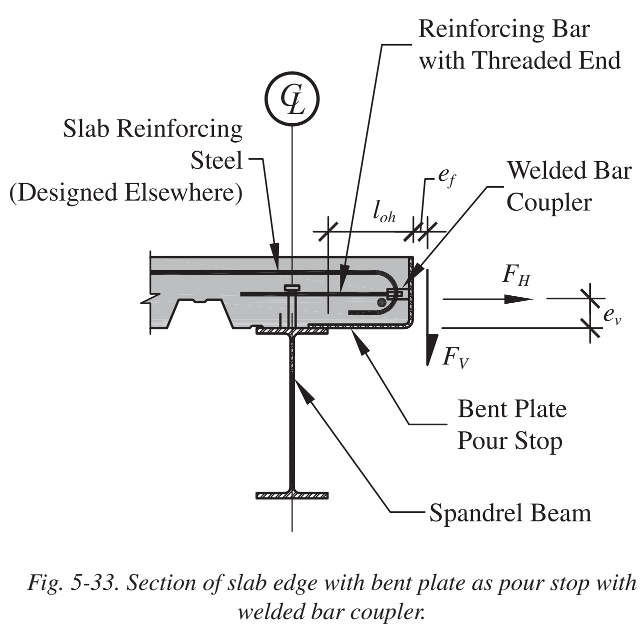


Fig. 5-33. Section of slab edge with bent plate as pour stop with welded bar coupler.

exists due to the horizontal eccentricity, e_f, between the vertical load, F_H, and the face of the plate.

$$\begin{aligned} M_{ef} &= F_V e_f \\ &= 1.8 \text{ kips}(5 \text{ in.}) \\ &= 9.00 \text{ kip-in.} \end{aligned}$$

This moment is resolved into a couple with the tension force, T, transferred directly to the slab by the welded bar coupler and threaded reinforcing bar, and the compression force, C, resisted by the plate in compression, as shown in Figure 5-34. With the welded bar coupler located at the mid-depth of the slab, the moment arm for this couple, e_v = 3¾ in. Thus, the bent plate can be designed for the flexure due to F_v applied at its tip only.

Try a 7⁄16-in. bent plate thickness. As calculated previously,

b_{eff} = 18.0 in., I = 0.126 in.^{4}, and Z = 0.861 in.^{3}

For the strength load combination 1.4D, the required flexural strength is,

$$\begin{aligned} M_u &= 1.4 F_V l_{oh} \\ &= 1.4(1.8 \text{ kips})(9 \text{ in.}) \\ &= 22.7 \text{ kip-in.} \end{aligned}$$

From AISC *Specification* Section F11, the available flexural strength is,

$$\begin{aligned} \phi_b M_n &= \phi_b F_y Z \\ &= 0.90(36\text{ksi})\left(0.861 \text{ in.}^3\right) \\ &= 27.9 \text{ kip-in.} \geq M_u \quad \textbf{o.k.} \end{aligned}$$

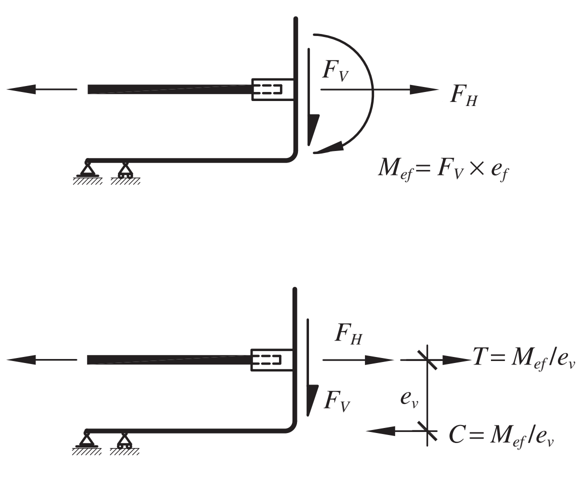


Fig. 5-34. Analysis model for plate design.

The 1.6F_yS limit need not be checked because Z/S = 1.5 for a rectangular plate in bending.

For deflection, using AISC *Manual* Table 3-23, case 26,

$$\begin{aligned} \Delta_V &= \frac{F_V l_{oh}^2}{3EI}\left(l_{oh} + s_{weld}\right) \\ &= \frac{1.8 \text{ kips}(9 \text{ in.})^2}{3(29{,}000 \text{ ksi})(0.126 \text{ in.}^4)}\left(9 \text{ in.} + 2\tfrac{1}{2} \text{ in.}\right) \\ &= 0.153 \text{ in.} \leq \tfrac{3}{16} \text{ in. } \textbf{o.k.} \end{aligned}$$

Use a 7⁄16-in. bent plate thickness.

Try a 3⁄16-in. fillet weld with F_{EXX} = 70 ksi between the bent plate pour stop and the spandrel beam flange. As developed previously in Example 5.3, the required length of fillet weld per ft of width is,

$$\begin{aligned} l_{min} &= \frac{M_u}{1.392 \text{ kip/in.}(1.5) D\ s_{weld}} \\ &= \frac{22.7 \text{ kip-in.}}{1.392 \text{ kip/in.}(1.5)(3)\left(2\tfrac{1}{2} \text{ in.}\right)} \\ &= 1.45 \text{ in.} \end{aligned}$$

The minimum permissible fillet weld length is 1½ in. per *Specification* Section J2.2b.

Use intermittent 3⁄16-in. fillet welds, 1½-in. long at 12 in. on center.

Comments:
Using a welded bar coupler and threaded reinforcing bar to resist the out-of-plane forces can reduce the required thickness of the bent plate pour stop, even in instances where the slab is not designed to resist the flexure as a cantilever. As illustrated in Figure 5-34, the welded bar coupler and threaded reinforcing bar must be designed for the effects of the tension forces. If the applicable load combination were 1.2D + 1.6W, the corresponding required strength is,

$$\begin{aligned} T_u + F_{u\,H} &= \frac{1.2 F_V e_f}{e_v} + 1.6 F_H \\ &= \frac{1.2(1.8 \text{ kips})(5 \text{ in.})}{3\tfrac{3}{4} \text{ in.}} + 1.6(1.8 \text{ kips}) \\ &= 5.76 \text{ kips} \end{aligned}$$

Headed studs can also be used for this purpose, as illustrated in Example 5.4.

As illustrated in Figure 5-34, there is also a compression component equal to,

$$C_u = \frac{1.2F_V e_f}{e_v}$$

$$= \frac{1.2(1.8 \text{ kips})(5 \text{ in.})}{3\tfrac{3}{4} \text{ in.}}$$

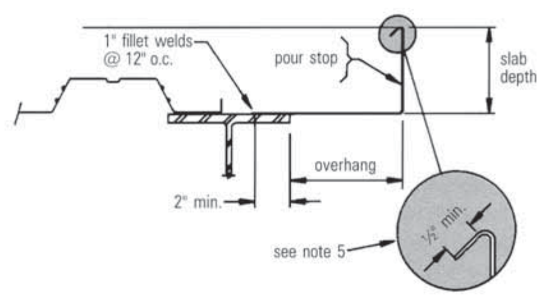
$$= 2.88 \text{ kips}$$

It is assumed that this force is resisted by bearing of the bent plate on the concrete. When this is not the case, or the concrete compressive strength is not adequate, the bent plate thickness can be selected based upon interaction of the moment and compressive force.

Table 5-1. Light-Gage Metal Pour Stop Selection Table for Normal-Weight Concrete

SLAB DEPTH (INCHES)	OVERHANG (INCHES)												
	0	1	2	3	4	5	6	7	8	9	10	11	12
	POUR STOP TYPES												
4.00	20	20	20	20	18	18	16	14	12	12	12	10	10
4.25	20	20	20	18	18	16	16	14	12	12	12	10	10
4.50	20	20	20	18	18	16	16	14	12	12	12	10	10
4.75	20	20	18	18	16	16	14	14	12	12	10	10	10
5.00	20	20	18	18	16	16	14	14	12	12	10	10	
5.25	20	18	18	16	16	14	14	12	12	12	10	10	
5.50	20	18	18	16	16	14	14	12	12	12	10	10	
5.75	20	18	16	16	14	14	12	12	12	12	10	10	
6.00	18	18	16	16	14	14	12	12	12	10	10	10	
6.25	18	18	16	14	14	12	12	12	12	10	10		
6.50	18	16	16	14	14	12	12	12	12	10	10		
6.75	18	16	14	14	14	12	12	12	10	10	10		
7.00	18	16	14	14	12	12	12	12	10	10	10		
7.25	16	16	14	14	12	12	12	10	10	10			
7.50	16	14	14	12	12	12	12	10	10	10			
7.75	16	14	14	12	12	12	10	10	10	10			
8.00	14	14	12	12	12	12	10	10	10				
8.25	14	14	12	12	12	10	10	10	10				
8.50	14	12	12	12	12	10	10	10					
8.75	14	12	12	12	12	10	10	10					
9.00	14	12	12	12	10	10	10						
9.25	12	12	12	12	10	10	10						
9.50	12	12	12	10	10	10							
9.75	12	12	12	10	10	10							
10.00	12	12	10	10	10	10							
10.25	12	12	10	10	10								
10.50	12	12	10	10	10								
10.75	12	10	10	10									
11.00	12	10	10	10									
11.25	12	10	10										
11.50	10	10	10										
11.75	10	10											
12.00	10	10											

TYPES	DESIGN THICKNESS
20	0.0358
18	0.0474
16	0.0598
14	0.0747
12	0.1046
10	0.1345



NOTES: This Selection Chart is based on following criteria:
1. Normal weight concrete (150 PCF).
2. Horizontal and vertical deflection is limited to ¼" maximum for concrete dead load.
3. Design stress is limited to 20 KSI for concrete dead load temporarily increased by one-third for the construction live load of 20 PSF.
4. Pour Stop Selection Chart does not consider the effect of the performance, deflection, or rotation of the pour stop support which may include both the supporting composite deck and/or the frame.
5. Vertical leg return lip is recommended for all types (gages).
6. This selection is not meant to replace the judgement of experienced Structural Engineers and shall be considered as a reference only.

SDI reserves the right to change any information in this selection table without notice.

Published by the Steel Deck Institute (SDI, 2000). Used with permission from the Steel Deck Institute.

Table 5-2. Cantilevered Slab Flexural Strength, ϕM_n, kip-in./ft
Concrete Compressive Strength $f'_c = 3{,}000$ psi
2-in. Composite Floor Deck Parallel to Spandrel Beam

Slab Reinforcement		Total Slab Height, h_s, in.										
Bars	**in.²/ft**	**4**	**4½**	**5**	**5¼ (6)**	**5½**	**6**	**6 3⁄16 (8)**	**6½ (7)**	**7**	**7¼ (9)**	**7½**
#3@18 (5)	0.0733	2.92	4.89	6.86	7.85	8.83	10.8	11.5	12.8	14.7	15.7	16.7
#3@16 (5)	0.0825	3.28	5.52	7.76	8.88	10.0	12.2	13.1	14.5	16.7	17.8	19.0
#3@12	0.110	4.18	7.15	10.1	11.6	13.1	16.1	17.2	19.0	22.0	23.5	25.0
#4@18 (5)	0.133	4.19	8.04	11.6	13.4	15.2	18.8	20.2	22.4	26.0	27.8	29.6
#4@16 (5)	0.150		8.93	13.0	15.0	17.0	21.1	22.6	25.1	29.2	31.2	33.2
#3@8	0.165		10.2	14.7	16.9	19.2	23.6	25.3	28.1	32.5	34.8	37.0
#4@12	0.200		11.4	16.8	19.5	22.2	27.6	29.6	33.0	38.4	41.1	43.8
#5@18 (5)	0.207		10.5	16.6	19.4	22.2	27.8	29.9	33.4	38.9	41.7	44.5
#3@6	0.220		12.8	19.0	21.9	24.9	30.8	33.1	36.8	42.7	45.7	48.7
#5@16	0.233			18.3	21.5	24.6	30.9	33.3	37.2	43.5	46.7	49.8
#4@8	0.300			22.7	27.6	31.7	39.8	42.8	47.9	56.0	60.0	64.1
#5@12	0.310			21.2	27.3	31.5	39.9	43.0	48.3	56.6	60.8	65.0
#4@6	0.400					37.7	50.9	55.0	61.7	72.5	77.9	83.3
#5@8	0.465						53.7	60.7	68.6	81.1	87.4	93.7
#5@6	0.620									101	111	120

Notes:
1. Dark shading indicates that the reinforcement ratio is such that the strain in the tensile reinforcement is less than 0.004 at nominal strength. This condition is prohibited for nonprestressed flexural members by ACI 318-05 Section 10.3.5.
2. Light shading indicates that reinforcement is less than minimum reinforcement ratio of 0.0018 required by ACI 318-05. This condition may be acceptable if the flexural strength exceeds the demand by more than one-third.
3. Top cover on bars is 1 in.
4. Flexural reinforcement must be adequately developed at point of maximum moment.
5. Bar spacing is greater than permitted by ACI 318-05 Section 10.6.4.
6. Slab thickness provides a 2-hour fire-rated assembly when used with lightweight concrete without fireproofing.
7. Slab thickness provides a 2-hour fire-rated assembly when used with normal-weight concrete without fireproofing.
8. Slab thickness provides a 3-hour fire-rated assembly when used with lightweight concrete without fireproofing.
9. Slab thickness provides a 3-hour fire-rated assembly when used with normal-weight concrete without fireproofing.
10. Steel reinforcement yield strength, $f_y = 60$ ksi.

Table 5-3. Cantilevered Slab Flexural Strength, ϕM_n, kip-in./ft
Concrete Compressive Strength f'_c = 4,000 psi
2-in. Composite Floor Deck Parallel to Spandrel Beam

Slab Reinforcement		Total Slab Height, h_s, in.										
Bars	**in.²/ft**	**4**	**4½**	**5**	**5¼ (6)**	**5½**	**6**	**6 3/16 (8)**	**6½ (7)**	**7**	**7¼ (9)**	**7½**
#3@18 (5)	0.0733	2.99	4.96	6.93	7.92	8.90	10.9	11.6	12.8	14.8	15.8	16.8
#3@16 (5)	0.0825	3.37	5.61	7.85	8.97	10.1	12.3	13.2	14.6	16.8	17.9	19.1
#3@12	0.110	4.34	7.31	10.3	11.8	13.3	16.2	17.3	19.2	22.2	23.6	25.1
#4@18 (5)	0.133	4.68	8.27	11.9	13.7	15.5	19.0	20.4	22.6	26.2	28.0	29.8
#4@16 (5)	0.150	5.18	9.23	13.3	15.3	17.3	21.4	22.9	25.4	29.5	31.5	33.5
#3@8	0.165	6.16	10.6	15.1	17.3	19.5	24.0	25.6	28.4	32.9	35.1	37.3
#4@12	0.200		11.9	17.3	20.0	22.7	28.1	30.1	33.5	38.9	41.6	44.3
#5@18 (5)	0.207		11.6	17.2	20.0	22.7	28.3	30.4	33.9	39.5	42.3	45.1
#3@6	0.220		13.7	19.6	22.6	25.5	31.5	33.7	37.4	43.4	46.3	49.3
#5@16	0.233		12.8	19.1	22.2	25.4	31.7	34.0	37.9	44.2	47.4	50.5
#4@8	0.300		15.5	24.8	28.8	32.9	41.0	44.0	49.1	57.2	61.2	65.3
#5@12	0.310			24.4	28.6	32.8	41.2	44.3	49.5	57.9	62.1	66.3
#4@6	0.400			30.3	36.8	42.2	53.0	57.1	63.8	74.6	80.0	85.4
#5@8	0.465				37.3	46.3	58.9	63.6	71.4	84.0	90.3	96.5
#5@6	0.620						71.5	80.9	91.4	108	117	125

Notes:
1. Dark shading indicates that the reinforcement ratio is such that the strain in the tensile reinforcement is less than 0.004 at nominal strength. This condition is prohibited for nonprestressed flexural members by ACI 318-05 Section 10.3.5.
2. Light shading indicates that reinforcement is less than minimum reinforcement ratio of 0.0018 required by ACI 318-05. This condition may be acceptable if the flexural strength exceeds the demand by more than one-third.
3. Top cover on bars is 1 in.
4. Flexural reinforcement must be adequately developed at point of maximum moment.
5. Bar spacing is greater than permitted by ACI 318-05 Section 10.6.4.
6. Slab thickness provides a 2-hour fire-rated assembly when used with lightweight concrete without fireproofing.
7. Slab thickness provides a 2-hour fire-rated assembly when used with normal-weight concrete without fireproofing.
8. Slab thickness provides a 3-hour fire-rated assembly when used with lightweight concrete without fireproofing.
9. Slab thickness provides a 3-hour fire-rated assembly when used with normal-weight concrete without fireproofing.
10. Steel reinforcement yield strength, f_y = 60 ksi.

Table 5-4. Cantilevered Slab Flexural Strength, ϕM_n, kip-in./ft
Concrete Compressive Strength f'_c = 3,000 psi
3-in. Composite Floor Deck Parallel to Spandrel Beam

Slab Reinforcement		Total Slab Height, h_s, in.										
Bars	**in.²/ft**	**5**	**5½**	**6**	**6¼ (6)**	**6½**	**7**	**7$\frac{3}{16}$ (8)**	**7½ (7)**	**8**	**8¼ (9)**	**8½**
#3@18 (5)	0.0733	2.92	4.89	6.86	7.84	8.83	10.8	11.5	12.8	14.7	15.7	16.7
#3@16 (5)	0.0825	3.28	5.52	7.76	8.88	10.0	12.2	13.1	14.5	16.7	17.8	19.0
#3@12	0.110	4.18	7.15	10.1	11.6	13.1	16.1	17.2	19.0	22.0	23.5	25.0
#4@18 (5)	0.133	4.19	8.04	11.6	13.4	15.2	18.8	20.2	22.4	26.0	27.8	29.6
#4@16 (5)	0.150		8.93	13.0	15.0	17.0	21.1	22.6	25.1	29.2	31.2	33.2
#3@8	0.165		10.2	14.7	16.9	19.2	23.6	25.3	28.1	32.5	34.8	37.0
#4@12	0.200		11.4	16.8	19.5	22.2	27.6	29.6	33.0	38.4	41.1	43.8
#5@18 (5)	0.207		10.5	16.6	19.4	22.2	27.8	29.9	33.4	38.9	41.7	44.5
#3@6	0.220		12.8	19.0	21.9	24.9	30.8	33.1	36.8	42.7	45.7	48.7
#5@16	0.233			18.3	21.5	24.6	30.9	33.3	37.2	43.5	46.7	49.8
#4@8	0.300			22.7	27.6	31.7	39.8	42.8	47.9	56.0	60.0	64.1
#5@12	0.310			21.2	27.3	31.5	39.9	43.0	48.3	56.6	60.8	65.0
#4@6	0.400					37.7	50.9	55.0	61.7	72.5	77.9	83.3
#5@8	0.465						53.7	60.7	68.6	81.1	87.4	93.7
#5@6	0.620									101	111	120

Notes:
1. Dark shading indicates that the reinforcement ratio is such that the strain in the tensile reinforcement is less than 0.004 at nominal strength. This condition is prohibited for nonprestressed flexural members by ACI 318-05 Section 10.3.5.
2. Light shading indicates that reinforcement is less than minimum reinforcement ratio of 0.0018 required by ACI 318-05. This condition may be acceptable if the flexural strength exceeds the demand by more than one-third.
3. Top cover on bars is 1 in.
4. Flexural reinforcement must be adequately developed at point of maximum moment.
5. Bar spacing is greater than permitted by ACI 318-05 Section 10.6.4.
6. Slab thickness provides a 2-hour fire-rated assembly when used with lightweight concrete without fireproofing.
7. Slab thickness provides a 2-hour fire-rated assembly when used with normal-weight concrete without fireproofing.
8. Slab thickness provides a 3-hour fire-rated assembly when used with lightweight concrete without fireproofing.
9. Slab thickness provides a 3-hour fire-rated assembly when used with normal-weight concrete without fireproofing.
10. Steel reinforcement yield strength, f_y = 60 ksi.

Table 5-5. Cantilevered Slab Flexural Strength, ϕM_n, kip-in./ft
Concrete Compressive Strength f'_c = 4,000 psi
3-in. Composite Floor Deck Parallel to Spandrel Beam

Slab Reinforcement		Total Slab Height, h_s, in.										
Bars	**in.^{2}/ft**	**5**	**5½**	**6**	**6¼ (6)**	**6½**	**7**	**7 3/16 (8)**	**7½ (7)**	**8**	**8¼ (9)**	**8½**
#3@18 (5)	0.0733	2.99	4.96	6.93	7.92	8.90	10.9	11.6	12.8	14.8	15.8	16.8
#3@16 (5)	0.0825	3.37	5.61	7.85	8.97	10.1	12.3	13.2	14.6	16.8	17.9	19.1
#3@12	0.110	4.34	7.31	10.3	11.8	13.3	16.2	17.3	19.2	22.2	23.6	25.1
#4@18 (5)	0.133	4.68	8.27	11.9	13.7	15.5	19.0	20.4	22.6	26.2	28.0	29.8
#4@16 (5)	0.150	5.18	9.23	13.3	15.3	17.3	21.4	22.9	25.4	29.5	31.5	33.5
#3@8	0.165	6.16	10.6	15.1	17.3	19.5	24.0	25.6	28.4	32.9	35.1	37.3
#4@12	0.200		11.9	17.3	20.0	22.7	28.1	30.1	33.5	38.9	41.6	44.3
#5@18 (5)	0.207		11.6	17.2	20.0	22.7	28.3	30.4	33.9	39.5	42.3	45.1
#3@6	0.220		13.7	19.6	22.6	25.5	31.5	33.7	37.4	43.4	46.3	49.3
#5@16	0.233		12.8	19.1	22.2	25.4	31.7	34.0	37.9	44.2	47.4	50.5
#4@8	0.300		15.5	24.8	28.8	32.9	41.0	44.0	49.1	57.2	61.2	65.3
#5@12	0.310			24.4	28.6	32.8	41.2	44.3	49.5	57.9	62.1	66.3
#4@6	0.400			30.3	36.8	42.2	53.0	57.1	63.8	74.6	80.0	85.4
#5@8	0.465				37.3	46.3	58.9	63.6	71.4	84.0	90.3	96.5
#5@6	0.620						71.5	80.9	91.4	108	117	125

Notes:
1. Dark shading indicates that the reinforcement ratio is such that the strain in the tensile reinforcement is less than 0.004 at nominal strength. This condition is prohibited for nonprestressed flexural members by ACI 318-05 Section 10.3.5.
2. Light shading indicates that reinforcement is less than minimum reinforcement ratio of 0.0018 required by ACI 318-05. This condition may be acceptable if the flexural strength exceeds the demand by more than one-third.
3. Top cover on bars is 1 in.
4. Flexural reinforcement must be adequately developed at point of maximum moment.
5. Bar spacing is greater than permitted by ACI 318-05 Section 10.6.4.
6. Slab thickness provides a 2-hour fire-rated assembly when used with lightweight concrete without fireproofing.
7. Slab thickness provides a 2-hour fire-rated assembly when used with normal-weight concrete without fireproofing.
8. Slab thickness provides a 3-hour fire-rated assembly when used with lightweight concrete without fireproofing.
9. Slab thickness provides a 3-hour fire-rated assembly when used with normal-weight concrete without fireproofing.
10. Steel reinforcement yield strength, f_y = 60 ksi.

Table 5-6. Cantilevered Slab Flexural Strength, ϕM_n, kip-in./ft Concrete Compressive Strength f'_c = 3,000 psi 2-in. Composite Floor Deck Perpendicular to Spandrel Beam												
Slab Reinforcement		**Total Slab Height, h_s, in.**										
Bars	**in.^{2}/ft**	**4**	**4½**	**5**	**5¼ (5)**	**5½**	**6**	**6$\frac{3}{16}$ (7)**	**6½ (6)**	**7**	**7¼ (8)**	**7½**
#3@18 (4)	0.0733	10.6	12.5	14.5	15.5	16.5	18.5	19.2	20.5	22.4	23.4	24.4
#3@16 (4)	0.0825	11.9	14.1	16.4	17.5	18.6	20.8	21.7	23.1	25.3	26.4	27.6
#3@12	0.110	15.4	18.4	21.4	22.8	24.3	27.3	28.4	30.3	33.2	34.7	36.2
#4@18 (4)	0.133	17.9	21.5	25.1	26.8	28.6	32.2	33.6	35.8	39.4	41.2	43.0
#4@16 (4)	0.150	19.9	23.9	28.0	30.0	32.0	36.1	37.6	40.1	44.2	46.2	48.2
#3@8	0.165	22.2	26.6	31.1	33.3	35.5	40.0	41.7	44.4	48.9	51.1	53.4
#4@12	0.200	25.5	30.9	36.3	38.9	41.7	47.1	49.1	52.5	57.9	60.6	63.3
#5@18 (4)	0.207	25.5	31.1	36.7	39.5	42.3	47.8	49.9	53.4	59.0	61.8	64.6
#3@6	0.220	28.3	34.2	40.2	43.1	46.1	52.0	54.3	58.0	63.9	66.9	69.9
#5@16	0.233	26.8	34.3	40.6	43.8	46.9	53.2	55.6	59.5	65.8	68.9	72.1
#4@8	0.300		39.7	51.2	55.2	59.3	67.4	70.4	75.5	83.6	87.6	91.7
#5@12	0.310			50.4	55.7	59.9	68.3	71.4	76.6	85.0	89.2	93.4
#4@6	0.400						83.6	89.7	96.4	107	113	118
#5@8	0.465								100	119	126	132
#5@6	0.620											

Notes:
1. Dark shading indicates that the reinforcement ratio is such that the strain in the tensile reinforcement is less than 0.004 at nominal strength. This condition is prohibited for nonprestressed flexural members by ACI 318-05 Section 10.3.5.
2. Top cover on bars is 1 in.
3. Flexural reinforcement must be adequately developed at point of maximum moment.
4. Bar spacing is greater than permitted by ACI 318-05 Section 10.6.4.
5. Slab thickness provides a 2-hour fire-rated assembly when used with lightweight concrete without fireproofing.
6. Slab thickness provides a 2-hour fire-rated assembly when used with normal-weight concrete without fireproofing.
7. Slab thickness provides a 3-hour fire-rated assembly when used with lightweight concrete without fireproofing.
8. Slab thickness provides a 3-hour fire-rated assembly when used with normal-weight concrete without fireproofing.
9. Steel reinforcement yield strength, f_y = 60 ksi.
10. The effective width of the compression zone is assumed to be 6 in. per ft of width based on typical composite floor deck profiles.

Table 5-7. Cantilevered Slab Flexural Strength, ϕM_n, kip-in./ft
Concrete Compressive Strength f'_c = 4,000 psi
2-in. Composite Floor Deck Perpendicular to Spandrel Beam

Slab Reinforcement		Total Slab Height, h_s, in.										
Bars	**in.^{2}/ft**	**4**	**4½**	**5**	**5¼ (5)**	**5½**	**6**	**6 3/16 (7)**	**6½ (6)**	**7**	**7¼ (8)**	**7½**
#3@18 (4)	0.0733	10.7	12.7	14.7	15.7	16.6	18.6	19.4	20.6	22.6	23.6	24.6
#3@16 (4)	0.0825	12.1	14.3	16.5	17.7	18.8	21.0	21.9	23.3	25.5	26.6	27.7
#3@12	0.110	15.7	18.7	21.7	23.2	24.7	27.6	28.7	30.6	33.6	35.1	36.5
#4@18 (4)	0.133	18.3	21.9	25.5	27.3	29.1	32.7	34.1	36.3	39.9	41.7	43.5
#4@16 (4)	0.150	20.5	24.5	28.6	30.6	32.6	36.7	38.2	40.7	44.8	46.8	48.8
#3@8	0.165	22.9	27.3	31.8	34.0	36.3	40.7	42.4	45.2	49.6	51.8	54.1
#4@12	0.200	26.5	31.9	37.3	40.0	42.7	48.1	50.1	53.5	58.9	61.6	64.3
#5@18 (4)	0.207	26.6	32.2	37.8	40.6	43.4	49.0	51.1	54.6	60.2	63.0	65.8
#3@6	0.220	29.6	35.5	41.4	44.4	47.4	53.3	55.5	59.3	65.2	68.2	71.1
#5@16	0.233	29.5	35.8	42.1	45.2	48.4	54.7	57.0	60.9	67.2	70.4	73.5
#4@8	0.300	37.2	45.5	53.6	57.6	61.7	69.8	72.8	77.9	86.0	90.0	94.1
#5@12	0.310	35.8	45.7	54.1	58.3	62.4	70.8	74.0	79.2	87.6	91.7	95.9
#4@6	0.400		54.2	68.3	73.7	79.1	89.9	93.9	101	112	117	122
#5@8	0.465				76.9	86.5	100	105	113	126	132	138
#5@6	0.620								134	159	168	177

Notes:
1. Dark shading indicates that the reinforcement ratio is such that the strain in the tensile reinforcement is less than 0.004 at nominal strength. This condition is prohibited for nonprestressed flexural members by ACI 318-05 Section 10.3.5.
2. Top cover on bars is 1 in.
3. Flexural reinforcement must be adequately developed at point of maximum moment.
4. Bar spacing is greater than permitted by ACI 318-05 Section 10.6.4.
5. Slab thickness provides a 2-hour fire-rated assembly when used with lightweight concrete without fireproofing.
6. Slab thickness provides a 2-hour fire-rated assembly when used with normal-weight concrete without fireproofing.
7. Slab thickness provides a 3-hour fire-rated assembly when used with lightweight concrete without fireproofing.
8. Slab thickness provides a 3-hour fire-rated assembly when used with normal-weight concrete without fireproofing.
9. Steel reinforcement yield strength, f_y = 60 ksi.
10. The effective width of the compression zone is assumed to be 6 in. per ft of width based on typical composite floor deck profiles.

Table 5-8. Cantilevered Slab Flexural Strength, ϕM_n, kip-in./ft
Concrete Compressive Strength $f'_c = 3{,}000$ psi
3-in. Composite Floor Deck Perpendicular to Spandrel Beam

Slab Reinforcement		Total Slab Height, h_s, in.										
Bars	**in.^{2}/ft**	**5**	**5½**	**6**	**6¼ (6)**	**6½**	**7**	**7 3⁄16 (8)**	**7½ (7)**	**8**	**8¼ (9)**	**8½**
#3@18 (5)	0.0733	14.5	16.5	18.5	19.5	20.5	22.4	23.2	24.4	26.4	27.4	28.4
#3@16 (5)	0.0825	16.4	18.6	20.8	22.0	23.1	25.3	26.2	27.6	29.8	30.9	32.0
#3@12	0.110	21.4	24.3	27.3	28.8	30.3	33.2	34.4	36.2	39.2	40.7	42.2
#4@18 (5)	0.133	25.1	28.6	32.2	34.0	35.8	39.4	40.8	43.0	46.6	48.4	50.2
#4@16 (5)	0.150	28.0	32.0	36.1	38.1	40.1	44.2	45.7	48.2	52.3	54.3	56.3
#3@8	0.165	31.1	35.5	40.0	42.2	44.4	48.9	50.6	53.4	57.8	60.0	62.3
#4@12	0.200	36.3	41.7	47.1	49.8	52.5	57.9	59.9	63.3	68.7	71.4	74.1
#5@18 (5)	0.207	36.7	42.3	47.8	50.6	53.4	59.0	61.1	64.6	70.2	73.0	75.8
#3@6	0.220	40.2	46.1	52.0	55.0	58.0	63.9	66.1	69.9	75.8	78.8	81.7
#5@16	0.233	40.6	46.9	53.2	56.4	59.5	65.8	68.2	72.1	78.4	81.5	84.7
#4@8	0.300	51.2	59.3	67.4	71.4	75.5	83.6	86.6	91.7	99.8	104	108
#5@12	0.310	50.4	59.9	68.3	72.4	76.6	85.0	88.1	93.4	102	106	110
#4@6	0.400			83.6	91.0	96.4	107	113	118	129	134	140
#5@8	0.465					100	119	125	132	145	151	158
#5@6	0.620									166	179	192

Notes:
1. Dark shading indicates that the reinforcement ratio is such that the strain in the tensile reinforcement is less than 0.004 at nominal strength. This condition is prohibited for nonprestressed flexural members by ACI 318-05 Section 10.3.5.
2. Light shading indicates that reinforcement is less than minimum reinforcement ratio of 0.0018 required by ACI 318-05. This condition may be acceptable if the flexural strength exceeds the demand by more than one-third.
3. Top cover on bars is 1 in.
4. Flexural reinforcement must be adequately developed at point of maximum moment.
5. Bar spacing is greater than permitted by ACI 318-05 Section 10.6.4.
6. Slab thickness provides a 2-hour fire-rated assembly when used with lightweight concrete without fireproofing.
7. Slab thickness provides a 2-hour fire-rated assembly when used with normal-weight concrete without fireproofing.
8. Slab thickness provides a 3-hour fire-rated assembly when used with lightweight concrete without fireproofing.
9. Slab thickness provides a 3-hour fire-rated assembly when used with normal-weight concrete without fireproofing.
10. Steel reinforcement yield strength, $f_y = 60$ ksi.
11. The effective width of the compression zone is assumed to be 6 in. per ft of width based on typical composite floor deck profiles.

Table 5-9. Cantilevered Slab Flexural Strength, ϕM_n, kip-in./ft Concrete Compressive Strength f'_c = 4,000 psi 3-in. Composite Floor Deck Perpendicular to Spandrel Beam												
Slab Reinforcement		**Total Slab Height, h_s, in.**										
Bars	**in.^{2}/ft**	**5**	**5½**	**6**	**6¼ (6)**	**6½**	**7**	**7$\frac{3}{16}$ (8)**	**7½ (7)**	**8**	**8¼ (9)**	**8½**
#3@18 (5)	0.0733	14.7	16.6	18.6	19.6	20.6	22.6	23.3	24.6	26.5	27.5	28.5
#3@16 (5)	0.0825	16.5	18.8	21.0	22.1	23.3	25.5	26.3	27.7	30.0	31.1	32.2
#3@12	0.110	21.7	24.7	27.6	29.1	30.6	33.6	34.7	36.5	39.5	41.0	42.5
#4@18 (5)	0.133	25.5	29.1	32.7	34.5	36.3	39.9	41.2	43.5	47.1	48.9	50.7
#4@16 (5)	0.150	28.6	32.6	36.7	38.7	40.7	44.8	46.3	48.8	52.9	54.9	56.9
#3@8	0.165	31.8	36.3	40.7	42.9	45.2	49.6	51.3	54.1	58.5	60.8	63.0
#4@12	0.200	37.3	42.7	48.1	50.8	53.5	58.9	60.9	64.3	69.7	72.4	75.1
#5@18 (5)	0.207	37.8	43.4	49.0	51.8	54.6	60.2	62.3	65.8	71.3	74.1	76.9
#3@6	0.220	41.4	47.4	53.3	56.3	59.3	65.2	67.4	71.1	77.1	80.1	83.0
#5@16	0.233	42.1	48.4	54.7	57.8	60.9	67.2	69.6	73.5	79.8	83.0	86.1
#4@8	0.300	53.6	61.7	69.8	73.8	77.9	86.0	89.0	94.1	102	106	110
#5@12	0.310	54.1	62.4	70.8	74.9	79.2	87.6	90.7	95.9	104	109	113
#4@6	0.400	68.3	79.1	89.9	95.3	101	112	116	122	133	139	144
#5@8	0.465		86.5	100	107	113	126	130	138	151	157	163
#5@6	0.620					134	159	166	177	193	202	210

Notes:

1. Dark shading indicates that the reinforcement ratio is such that the strain in the tensile reinforcement is less than 0.004 at nominal strength. This condition is prohibited for nonprestressed flexural members by ACI 318-05 Section 10.3.5.
2. Light shading indicates that reinforcement is less than minimum reinforcement ratio of 0.0018 required by ACI 318-05. This condition may be acceptable if the flexural strength exceeds the demand by more than one-third.
3. Top cover on bars is 1 in.
4. Flexural reinforcement must be adequately developed at point of maximum moment.
5. Bar spacing is greater than permitted by ACI 318-05 Section 10.6.4.
6. Slab thickness provides a 2-hour fire-rated assembly when used with lightweight concrete without fireproofing.
7. Slab thickness provides a 2-hour fire-rated assembly when used with normal-weight concrete without fireproofing.
8. Slab thickness provides a 3-hour fire-rated assembly when used with lightweight concrete without fireproofing.
9. Slab thickness provides a 3-hour fire-rated assembly when used with normal-weight concrete without fireproofing.
10. Steel reinforcement yield strength, f_y = 60 ksi.
11. The effective width of the compression zone is assumed to be 6 in. per ft of width based on typical composite floor deck profiles.

Table 5-10. Minimum Thickness of Bent Plate, in.
Used as a Pour Stop for Normal-Weight Concrete

Slab Thickness, in.	Slab Overhang, in.														
	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
4	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16
4¼	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16
4½	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16
4¾	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16
5	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16
5¼	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16
5½	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16
5¾	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16
6	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	3/8
6¼	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	5/16	5/16	5/16	3/8
6½	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16	3/8
6¾	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16	3/8
7	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16	3/8
7¼	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	3/8	3/8
7½	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	3/8	3/8
7¾	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	3/8	3/8
8	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16	3/8	3/8
8¼	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16	3/8	3/8
8½	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16	3/8	3/8
8¾	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	3/8	3/8	3/8
9	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	3/8	3/8	3/8
9¼	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	3/8	3/8	3/8
9½	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	3/8	3/8	7/16
9¾	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	5/16	5/16	5/16	3/8	3/8	7/16
10	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16	3/8	3/8	7/16

Notes:
1. Plate thickness is controlled by a maximum vertical deflection of ⅛ in. at the tip of the pour stop under the wet load of the concrete. If deflection is not a concern, a thinner plate may provide adequate strength.
2. Steel plate is ASTM A36 material.
3. Concrete unit weight is 150 lb/ft^3.
4. Pour stops are checked for strength considering the wet load of the concrete, a 250-lb point load applied at the tip of the pour stop and a uniform construction load of 20 psf. The point load and uniform construction load are not normally applied concurrently.
5. Shading is provided for visual clarity.

Table 5-11. Minimum Thickness of Bent Plate, in. Used as a Pour Stop for Lightweight Concrete

Slab Thickness, in.	Slab Overhang, in.														
	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18
4	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16
4¼	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16
4½	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	1/4	5/16
4¾	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16
5	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16
5¼	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16
5½	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16
5¾	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16
6	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	1/4	5/16	5/16
6¼	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16
6½	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16
6¾	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16
7	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16
7¼	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	3/8
7½	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	3/8
7¾	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16	3/8
8	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16	3/8
8¼	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16	3/8
8½	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16	3/8
8¾	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	3/8	3/8
9	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	3/8	3/8
9¼	3/16	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	5/16	5/16	5/16	3/8	3/8
9½	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16	3/8	3/8
9¾	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16	3/8	3/8
10	3/16	3/16	3/16	3/16	3/16	3/16	3/16	1/4	1/4	1/4	5/16	5/16	5/16	3/8	3/8

Notes:
1. Plate thickness is controlled by a maximum vertical deflection of 1/8 in. at the tip of the pour stop under the wet load of the concrete. If deflection is not a concern, a thinner plate may provide adequate strength.
2. Steel plate is ASTM A36 material.
3. Concrete unit weight is 115 lb/ft^3.
4. Pour stops are checked for strength considering the wet load of the concrete, a 250-lb point load applied at the tip of the pour stop, and a uniform construction load of 20 psf. The point load and uniform construction load are not normally applied concurrently.
5. Shading is provided for visual clarity.

Table 5-12. Headed Stud Tensile Strength, ϕN_n, kips 4,000 psi Normal-Weight Concrete Slab Edges											
Headed Stud Diameter, in.	**Embedment Depth, in.**	**Total Slab Height, h_s, in.**									
		4	**4½**	**5**	**5½**	**6**	**6½**	**7**	**7¼**	**7½**	**8¼**
½	**6**	2.66	3.03	3.40	3.78	4.16	4.56	4.96	5.16	5.30	5.99
	8	3.01	3.41	3.82	4.24	4.66	5.09	5.30	5.74	5.30	6.64
	10	3.32	3.75	4.20	4.65	5.11	5.30	5.30	6.27	5.30	7.23
⅝	**6**	2.66	3.03	3.40	3.78	4.16	4.56	4.96	5.16	5.37	5.99
	8	3.01	3.41	3.82	4.24	4.66	5.09	5.52	5.74	5.96	6.64
	10	3.32	3.75	4.20	4.65	5.11	5.57	6.04	6.27	6.51	7.23
¾	**6**	2.66	3.03	3.40	3.78	4.16	4.56	4.96	5.16	5.37	5.99
	8	3.01	3.41	3.82	4.24	4.66	5.09	5.52	5.74	5.96	6.64
	10	3.32	3.75	4.20	4.65	5.11	5.57	6.04	6.27	6.51	7.23

Notes:
1. Stud strengths are for single cast-in headed studs based on ACI 318-05 Appendix D. Refer to Section D.7 for interaction of tension and shear on headed studs.
2. In regions of moderate or high seismicity, values must be multiplied by 0.75 per Section D.3.3.3 of ACI 318-05 Appendix D.
3. Stud material is assumed to have a tensile strength of 60 ksi.
4. Calculations assume that the stud is centered at the mid-thickness of the slab, and that there are no free edges along the length of the slab within a distance of 1.5 times the embedment depth of the stud measured from the center of the stud.

Table 5-13. Headed Stud Shear Strength, ϕV_n, kips 4,000 psi Normal-Weight Concrete Slab Edges											
Headed Stud Diameter, in.	**Embedment Depth, in.**	**Total Slab Height, h_s, in.**									
		4	**4½**	**5**	**5½**	**6**	**6½**	**7**	**7¼**	**7½**	**8¼**
½	**6**	1.07	1.28	1.50	1.73	1.97	2.22	2.49	2.62	2.76	3.18
	8	1.07	1.28	1.50	1.73	1.97	2.22	2.49	2.62	2.76	3.18
	10	1.07	1.28	1.50	1.73	1.97	2.22	2.49	2.62	2.76	3.18
⅝	**6**	1.20	1.43	1.68	1.94	2.21	2.49	2.78	2.93	3.08	3.56
	8	1.20	1.43	1.68	1.94	2.21	2.49	2.78	2.93	3.08	3.56
	10	1.20	1.43	1.68	1.94	2.21	2.49	2.78	2.93	3.08	3.56
¾	**6**	1.31	1.57	1.84	2.12	2.42	2.72	3.04	3.21	3.38	3.89
	8	1.31	1.57	1.84	2.12	2.42	2.72	3.04	3.21	3.38	3.89
	10	1.31	1.57	1.84	2.12	2.42	2.72	3.04	3.21	3.38	3.89

Notes:
1. Stud strengths are for single cast-in headed studs welded to steel attachments with a ⅜-in. minimum thickness based on ACI 318-05 Appendix D. Refer to Section D.7 for interaction of tension and shear on headed studs.
2. In regions of moderate or high seismicity, values must be multiplied by 0.75 per Section D.3.3.3 of ACI 318-05 Appendix D.
3. Stud material is assumed to have a tensile strength of 60 ksi.
4. Calculations assume that the stud is centered at the mid-thickness of the slab, and that there are no free edges along the length of the slab within a distance of 1.5 times the embedment depth of the stud measured from the center of the stud.

Table 5-14. Headed Stud Tensile Strength, ϕN_n, kips 4,000 psi Lightweight Concrete Slab Edges													
Headed Stud Diameter, in.	**Embedment Depth, in.**	**Total Slab Height, h_s, in.**											
		4	**4½**	**5**	**5¼**	**5½**	**6**	**6³⁄₁₆**	**6¼**	**6½**	**7**	**7³⁄₁₆**	**7½**
½	**6**	2.00	2.27	2.55	2.69	2.83	3.12	3.23	3.27	3.42	3.72	3.83	4.03
	8	2.25	2.56	2.86	3.02	3.18	3.49	3.61	3.65	3.82	4.14	4.27	4.47
	10	2.49	2.82	3.15	3.32	3.49	3.83	3.96	4.00	4.18	4.53	4.66	4.88
⅝	**6**	2.00	2.27	2.55	2.69	2.83	3.12	3.23	3.27	3.42	3.72	3.83	4.03
	8	2.25	2.56	2.86	3.02	3.18	3.49	3.61	3.65	3.82	4.14	4.27	4.47
	10	2.49	2.82	3.15	3.32	3.49	3.83	3.96	4.00	4.18	4.53	4.66	4.88
¾	**6**	2.00	2.27	2.55	2.69	2.83	3.12	3.23	3.27	3.42	3.72	3.83	4.03
	8	2.25	2.56	2.86	3.02	3.18	3.49	3.61	3.65	3.82	4.14	4.27	4.47
	10	2.49	2.82	3.15	3.32	3.49	3.83	3.96	4.00	4.18	4.53	4.66	4.88

Notes:
1. Stud strengths are for single cast-in headed studs based on ACI 318-05 Appendix D. Refer to Section D.7 for interaction of tension and shear on headed studs.
2. In regions of moderate or high seismicity, values must be multiplied by 0.75 per Section D.3.3.3 of ACI 318-05 Appendix D.
3. Stud material is assumed to have a tensile strength of 60 ksi.
4. Calculations assume that the stud is centered at the mid-thickness of the slab, and that there are no free edges along the length of the slab within a distance of 1.5 times the embedment depth of the stud measured from the center of the stud.

Table 5-15. Headed Stud Shear Strength, ϕV_n, kips 4,000 psi Lightweight Concrete Slab Edges													
Headed Stud Diameter, in.	**Embedment Depth, in.**	**Total Slab Height, h_s, in.**											
		4	**4½**	**5**	**5¼**	**5½**	**6**	**6³⁄₁₆**	**6¼**	**6½**	**7**	**7³⁄₁₆**	**7½**
½	**6**	0.805	0.961	1.13	1.21	1.30	1.48	1.55	1.57	1.67	1.86	1.94	2.07
	8	0.805	0.961	1.13	1.21	1.30	1.48	1.55	1.57	1.67	1.86	1.94	2.07
	10	0.805	0.961	1.13	1.21	1.30	1.48	1.55	1.57	1.67	1.86	1.94	2.07
⅝	**6**	0.900	1.07	1.26	1.35	1.45	1.65	1.73	1.76	1.86	2.08	2.17	2.31
	8	0.900	1.07	1.26	1.35	1.45	1.65	1.73	1.76	1.86	2.08	2.17	2.31
	10	0.900	1.07	1.26	1.35	1.45	1.65	1.73	1.76	1.86	2.08	2.17	2.31
¾	**6**	0.986	1.18	1.38	1.48	1.59	1.81	1.90	1.93	2.04	2.28	2.38	2.53
	8	0.986	1.18	1.38	1.48	1.59	1.81	1.90	1.93	2.04	2.28	2.38	2.53
	10	0.986	1.18	1.38	1.48	1.59	1.81	1.90	1.93	2.04	2.28	2.38	2.53

Notes:
1. Stud strengths are for single cast-in headed studs welded to steel attachments with a ⅜-in. minimum thickness based on ACI 318-05 Appendix D. Refer to Section D.7 for interaction of tension and shear on headed studs.
2. In regions of moderate or high seismicity, values must be multiplied by 0.75 per Section D.3.3.3 of ACI 318-05 Appendix D.
3. Stud material is assumed to have a tensile strength of 60 ksi.
4. Calculations assume that the stud is centered at the mid-thickness of the slab, and that there are no free edges along the length of the slab within a distance of 1.5 times the embedment depth of the stud measured from the center of the stud.

Chapter 6
Design of Steel Spandrel Beams

6.1 GENERAL DESIGN CONSIDERATIONS

The design of a spandrel beam involves more than just selecting a wide-flange shape that meets flexural strength and stiffness criteria. Flexural strength and stiffness are primary, but only two of potentially many other factors that must be considered, some of which are detailing requirements influenced by architectural and mechanical design decisions.

Common design considerations include, but may not be limited to, the following:

Flexural strength—Is the spandrel beam composite or noncomposite? Is it part of a moment frame? Is weak-axis bending from façade forces a design consideration due to large floor openings adjacent to the spandrel beam?

Flexural stiffness—What are the superimposed dead- and live-load deflection requirements? What are the façade deflection requirements? Do floor vibration considerations affect the design? Is long-term deflection from creep an issue in composite sections? Are lateral deflections from façade forces a design consideration due to large floor openings adjacent to the spandrel beam?

Connections to columns—Are they simple shear connections or are the spandrel beams part of a moment frame? Are they typical in configuration, or is the web of the spandrel beam offset from the column centerline enough that nonstandard connections are necessary? Are horizontal forces a design consideration? Does the connection geometry interfere with slab edge detailing requirements?

Torsion—Is torsion a design consideration? Is the torsion resolved at the columns, or with roll beams or kickers? Does the design address rotation of the spandrel beam, which results in rotation and projected vertical displacement of façade supports that are eccentric to the spandrel beam?

Depth—Are there depth requirements or limitations? Does the depth interfere with the window heads, shade pockets, or other features unique to the building perimeter conditions? Should any of the spandrel beams be deeper than structurally required to facilitate window head anchorage or other façade connections? Should a constant depth be used for the spandrel beams throughout the project for a standardized architectural or façade connection detail?

Flange width and thickness—Are there minimum or maximum flange dimensions required for the slab edge or façade attachment details? Is there enough width to allow for edge plates, headed studs (including the size of the installation ferrule), deck bearing, and other items? Is the flange thick enough to allow headed studs to be placed off the centerline, if necessary? Should the range of flange widths be minimized for the spandrel beams throughout the project?

Location—Is the centerline of the spandrel beam on the centerline of the columns or is it offset? Is the spandrel beam sufficiently close to the slab edge to minimize eccentricities from the façade? Is the spandrel beam sufficiently set back from the slab edge to allow for fire-safing, fireproofing, and façade elements to pass by with appropriate clearances for installation?

Fabrication and erection—Does the spandrel beam design appropriately account for fabrication and erection? Does the detail appropriately balance member weight with simplicity and efficiency of fabrication and erection?

Value engineering—Will the spandrel beam detail be easy to change if there is a change to the façade system as a result of value engineering? It is the author's experience that less-than-ideal spandrel beam details often result from last-minute (but necessary) cost reductions achieved through changing the façade system. Although designers cannot predict change with certainty, it is worth considering if there is potential for change and what may be the likely alternative façade systems.

Some of these considerations are unique to the spandrel beam, and many require the designer to evaluate the spandrel beam detail more closely than other project details. Some may also require give and take for coordination with architectural finishes and other building systems.

6.2 DESIGN OF THE SPANDREL BEAM FOR VERTICAL LOADS

For spandrel beams supporting façade loads, the design is often controlled by deflections unless the spandrel beam is part of a moment frame. Other AISC publications, such as the *Specification for Structural Steel Buildings* (AISC, 2005b), hereafter referred to as the AISC Specification, provide guidance on the strength design of composite and noncomposite steel beams. However, the following are potential issues the designer should be aware of when evaluating the strength of spandrel beams for vertical loads:

- Do slab edge closure plates and/or other façade attachment elements in the slab preclude proper spacing of headed studs?

- Do slab depressions or openings preclude designing the beam as a composite beam?

- Are there torsional effects or flexural effects from horizontal forces that must be combined with flexural effects from vertical loads?

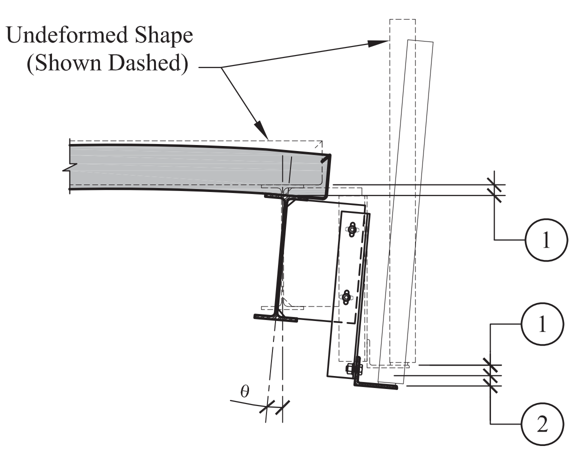
- Are there local strength issues from façade attachments, such as local web yielding or flange bending?

Deflection and movement limitations for most façade systems are such that the stiffness, not strength, will usually control the size of the spandrel beam. The deflection criteria for spandrel beams may not be simply a function of the span length as is often the case for floor beams. The applicable deflection or movement limitation(s) may be absolute-magnitude value(s), not a function of the span, for certain façade movement joint designs, such as horizontal soft joints in masonry veneers or precast-concrete panels.

The allowable spandrel beam vertical deflection is the portion of the allowable joint movement reserved for the structural deflection, less any significant deflections due to the flexibility of secondary support elements, such as hangers, relieving angles, and brackets. In addition, any rotation of the spandrel beam will translate into vertical and horizontal movements at the joint, as illustrated in Figure 6-1, further limiting the allowable vertical deflections.

After determining the allowable vertical deflections of the spandrel beam, the designer should consider when the spandrel beam deflections occur relative to the construction of the façade system. At least 50 percent of the floor live load deflections must be included in the design of the façade movement joints. However, significant superimposed floor dead loads may also require consideration. For example, the spandrel beam deflections due to the installation of stone or brick veneer will affect the design of the back-up anchorage and movement joints. For another example, the controlling deflection criteria for a spandrel beam supporting a brick veneer system may be the allowable deflection from the sum of all loads applied after the erection of the back-up, including superimposed dead and live loads, plus the weight of the veneer itself. These loads contribute to deflections that close the horizontal movement joint in the veneer.

Examples of calculated deflections for spandrel beams supporting four different façade systems are provided in Tables 6-1 through 6-4, which cover brick veneer cladding with 8-in. concrete masonry unit (CMU) back-up, brick veneer cladding with 6-in. metal stud back-up, 6-in. precast-concrete panel cladding, and aluminum curtain wall cladding, respectively. For each system, the table provides three spandrel beam sizes: the size if there were no deflection limits; the size if the deflections limits were similar to those used for typical floor beams, with a net total deflection less than L/240 and live load deflection less than L/360; and the size required to meet specific deflection limits that are typical for the particular façade system. In all four examples, the spandrel beam size is controlled by façade deflection limits with the most significant effect being for the brick veneer example, which has the most stringent limitations in these examples. Note that the same size spandrel beam is needed for both the CMU back-up and metal stud back-up systems despite the block being heavier. The controlling criterion is the allowable deflection for the joint, and both systems have the same load for this criterion.



NOTES:

1 Vertical deflection.

2 Added deflection due to rotation.

Fig. 6-1. Impact of angular rotation of spandrel beam on façade movement.

Table 6-1. Example Composite Spandrel Beam Deflections Brick Veneer Cladding 8-in. CMU Back-up (Grouted 32 in. o.c.)			
	Deflection Restriction		
	no δ restriction	$\delta_{NTD} = L/240 = 1\frac{1}{2}$ in. $\delta_{LL} = L/360 = 1$ in.	$\delta_{SIDL+LL} = \frac{3}{16}$ in.
Required spandrel size	W16x26	W16x31	W21x62
Pre-composite deflection, in.	0.606	0.494	0.154
Deflection due to back-up, in.	0.424	0.356	0.165
Deflection due to brick, in.	0.339	0.284	0.132
Deflection due to SIDL, in.	0.0389	0.0326	0.015
Deflection due to LL, in.	0.389	0.326	0.151
Camber, in.	none	none	none
Net Total Deflection (NTD), in.	1.80	1.49	0.617
$\delta_{SIDL+LL}$, **in.**	0.429	0.359	0.166
$M_u/\phi_b M_n$	0.934	0.774	0.421

Notes:

1. Superimposed Dead Load (SIDL) is the portion of the dead load applied after the concrete slab has cured, excluding cladding loads.
2. LL is the Live Load.
3. The Net Total Deflection (NTD) is the sum of the individual deflections minus the camber.
4. Camber is upward (opposite in direction to the in-service deflections).

Design Parameters:

Beam span	30 ft
SIDL	10 psf
LL	100 psf
Floor-to-floor height	12 ft
Floor tributary width	5½ ft
Slab construction	6¼-in. lightweight concrete on 3-in. deck
Brick veneer weight	40 psf
CMU weight	50 psf

Table 6-2. Example Composite Spandrel Beam Deflections Brick Veneer Cladding with 6-in. Metal Stud Back-up (16-in. Spacing)			
	Deflection Restriction		
	no δ restriction	$\delta_{NTD} = L/240 = 1\frac{1}{2}$ in. $\delta_{LL} = L/360 = 1$ in.	$\delta_{SIDL+LL} = \frac{3}{16}$ in.
Required spandrel size	W12x19	W16x26	W21x62
Pre-composite deflection, in.	1.37	0.606	0.154
Deflection due to back-up, in.	0.113	0.081	0.0297
Deflection due to brick, in.	0.501	0.359	0.132
Deflection due to SIDL, in.	0.0574	0.0411	0.0151
Deflection due to LL, in.	0.574	0.411	0.151
Camber, in.	None	none	none
Net Total Deflection (NTD), in.	2.62	1.50	0.482
$\delta_{SIDL+LL}$, **in.**	1.13	0.811	0.166
$M_u/\phi_b M_n$	0.937	0.754	0.327

Notes:

1. Superimposed Dead Load (SIDL) is the portion of the dead load applied after the concrete slab has cured, excluding cladding loads.
2. LL is the Live Load.
3. The Net Total Deflection (NTD) is the sum of the individual deflections minus the camber.
4. Camber is upward (opposite in direction to the in-service deflections).

Design Parameters:

Beam span	30 ft
SIDL	10 psf
LL	100 psf
Floor-to-floor height	12 ft
Floor tributary width	5½ ft
Slab construction	6¼-in. lightweight concrete on 3-in. deck
Brick veneer weight	40 psf
Metal stud weight	9 psf

Table 6-3. Example Composite Spandrel Beam Deflections 6-in. Precast-Concrete Panel Cladding			
	Deflection Restriction		
	no δ restriction	$\delta_{NTD} = L/240 = 1\frac{1}{2}$ in. $\delta_{LL} = L/360 = 1$ in.	$\delta_{SIDL+LL} = \frac{3}{8}$ in.
Required spandrel size	W14x22	W16x26	W18x40
Pre-composite deflection, in.	0.903	0.606	0.312
Deflection due to precast, in.	0.735	0.533	0.457
Deflection due to SIDL, in.	0.0449	0.0326	0.0297
Deflection due to LL, in.	0.449	0.326	0.297
Camber, in.	none	none	none
Net Total Deflection (NTD), in.	2.13	1.50	1.08
$\delta_{SIDL+LL}$, **in.**	1.23	0.892	0.307
$M_u/\phi_b M_n$	0.930	0.736	0.621

Notes:

1. Superimposed Dead Load (SIDL) is the portion of the dead load applied after the concrete slab has cured, excluding cladding loads.
2. LL is the Live Load.
3. The Net Total Deflection (NTD) is the sum of the individual deflections minus the camber.
4. Camber is upward (opposite in direction to the in-service deflections).

Design Parameters:

Beam span	30 ft
SIDL	10 psf
LL	100 psf
Floor-to-floor height	12 ft
Floor tributary width	5½ ft
Slab construction	6¼-in. lightweight concrete on 3-in. deck
Precast weight	75 psf

Table 6-4. Example Composite Spandrel Beam Deflections Aluminum Curtain Wall Cladding			
	Deflection Restriction		
	no δ restriction	$\delta_{NTD} = L/240 = 1\frac{1}{2}$ in. $\delta_{LL} = L/360 = 1$ in.	$\delta_{CW+SIDL+LL} = \frac{3}{4}$ in.
Required spandrel size	W12x14	W16x26	W16x26
Pre-composite deflection, in.	1.97	0.606	0.606
Deflection due to curtain wall, in.	0.243	0.151	0.151
Deflection due to SIDL, in.	0.0744	0.0462	0.0462
Deflection due to LL, in.	0.744	0.462	0.462
Camber, in.	1.25	none	none
Net Total Deflection (NTD), in.	1.78	1.27	1.27
$\delta_{cw+SIDL+LL}$, **in.**	1.06	0.659	0.659
$M_u/\phi_b M_n$	0.930	0.633	0.633

Notes:

1. Superimposed Dead Load (SIDL) is the portion of the dead load applied after the concrete slab has cured, excluding cladding loads.
2. LL is the Live Load.
3. The Net Total Deflection (NTD) is the sum of the individual deflections minus the camber.
4. Camber is upward (opposite in direction to the in-service deflections).

Design Parameters:

Beam span	30 ft
SIDL	10 psf
LL	100 psf
Floor-to-floor height	12 ft
Floor tributary width	5½ ft
Slab construction	6¼-in. lightweight concrete on 3-in. deck
Curtain wall weight	15 psf

For these typical examples, camber in the spandrel beam does not help reduce the size, except for the relatively light aluminum curtain wall system. Any camber in the spandrel beam also places greater tolerance demands on the façade attachments to account for camber tolerances and the uncertainty in the prediction of how much camber will actually come out after loading. Hence, there is wisdom in the common recommendation that camber should be avoided in spandrel beams.

As the examples in Tables 6-1 through 6-4 show, keeping the deflections of the spandrel beam small that occur after the sealant is installed in masonry veneers can lead to deep and heavy sections. This may impact the project when the overall steel tonnage of the building is a critical design objective. Communication between the engineer and architect is necessary to be sure the architect understands the impact the joint size has on the spandrel. Rather than determine the incremental deflections due to the live loads, superimposed dead loads, and brick loads, some engineers design spandrel beams supporting brick veneer walls for the lesser of *L*/600 or 0.31 in. as required by ACI 530-05 (ACI, 2005). If the architect's joint size is not consistent with the net deflection after the sealant is in place, the sealant's allowable repetitive strains may be exceeded as the brick's volume changes over time. This may lead to bulging and protruding sealant in the soft joint below the shelf angle and may shorten the interval at which the sealant needs to be replaced. The owner and design team should decide on a strategy that balances the up-front costs of additional steel tonnage against future maintenance costs.

6.3 DESIGN OF THE SPANDREL BEAM FOR TORSION

Some façade details and conditions tend to induce torsion on the spandrel beam, as illustrated in Figure 6-2. The rotational flexibility of a wide-flange shape when twisted about its longitudinal axis is such that torsion must be eliminated, if possible, or limited by providing torsional restraints along the length of the spandrel beam. Restraint is provided by kickers, floor beams or roll beams, and/or connections to columns.

If the location of restraints along the spandrel beam can be coordinated with the location of the applied façade loads that cause the torsion, torsion in the spandrel beam can be avoided altogether. However, for some façade systems, especially those designed by the façade contractor, the designer cannot be certain of the locations of the applied loads when designing the spandrel beams. Therefore, it is often prudent to consider the effect of eccentric façade loads applied between torsional restraints when selecting the spandrel beams. Economical designs are achieved when the spacing of restraint locations is between 10 and 15 ft on center, even if the spandrel beam is required to be slightly heavier for torsion applied between restraints.

Concrete slabs that are anchored to the spandrel beam top flange with shear stud connectors are an additional source

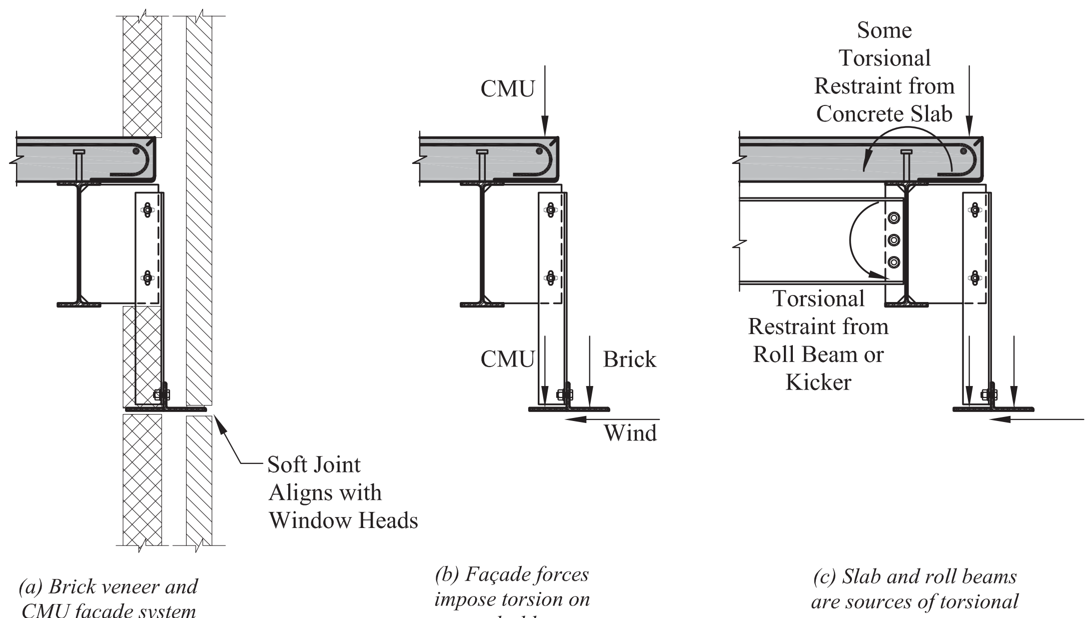


Fig. 6-2. Example detail that induces torsion on spandrel beam.

of torsional restraint for spandrel beams. The amount of rotational restraint offered by the slab depends on the flexural rigidity and strength of the slab. Strength and rigidity may not be adequate when the slab is composed of composite deck and the deck runs parallel with the spandrel beam, in which case it is common to not rely on the flexural strength of the composite slab as a sole means of restraining torsion. Even in this case, however, the designer can use the slab to reduce the rotation of the beam because the slab restrains lateral movement of the top flange. When the top flange is restrained from lateral movement in the plane of the slab, the center of rotation of the beam is at the top flange, as illustrated in Figure 6-3.

References such as AISC Design Guide No. 9, *Torsional Analysis of Structural Steel Members* (Seaburg and Carter, 1997), and Salmon and Johnson (1996) provide guidance on the design of wide-flange beams subjected to torsion. However, few references address the case of a beam with the top flange braced against translation by a deck or slab and subjected to torsion, which is the case for most spandrel beam conditions. The rotational response in this case is considerably different from that of a bare beam.

A designer may be inclined to treat the spandrel beam as a bare steel member, ignoring the slab entirely. While this generally will be conservative with respect to the total rotation, it does not consider that the beam rotates about the top flange. If the cladding attachment is at or below the center of gravity of the beam, the distance to the point of rotation, or radius, will be increased, as will the translational deflections at the cladding attachment.

The designer conservatively can account for the effects of the braced top flange by using the rotation obtained by the method presented in Seaburg and Carter (1997), but with the idealized rotation applied about the top flange of the spandrel beam when computing translation deflections of cladding supports.

Another option is to use the "flexural analogy" method presented by Salmon and Johnson (1996) for the analysis of steel beams in torsion. This method converts the applied twist into a force couple acting on the top and bottom flanges of the beam. The flanges are then considered as simply supported beams spanning between the torsional restraints in the real beam. As illustrated in Figure 6-4, the deflections are calculated with the bottom flange deflecting in one direction and the top flange deflecting in the opposite direction, and converting the result into an equivalent rotation. This approach can also account for the braced top flange by recognizing that the top flange does not translate and the beam rotates about the top. The bottom flange deflection is calculated and the rotation is determined using the full depth of the beam as the radius.

Appendix A presents a study that compares the methods just described to finite element models to assess their appropriateness for practical design. The study indicates that both approximate methods predict bottom flange deflections greater than those determined by finite element modeling. Of the two methods, the flexural analogy provides the most accurate prediction of the beam bottom flange lateral displacement for torsional spans in the range of 10 ft. At spans in the 20- to 30-ft range, the unrestrained rotation calculated using the method presented in Seaburg and Carter (1997), but applied relative to the top flange, provides a better approximation. See Appendix A for a discussion of the modeling procedure and a comparison of results.

A common condition in which the designer must consider torsion along a considerable length of spandrel beam is at floor openings along exterior walls, such as stairways, as illustrated in Figure 6-5. At these conditions, the spandrel beam often must be designed for weak-axis bending due to horizontal façade forces in addition to vertical loads and torsion over a significant length. It is generally uneconomical to increase the size of the spandrel beam for this loading

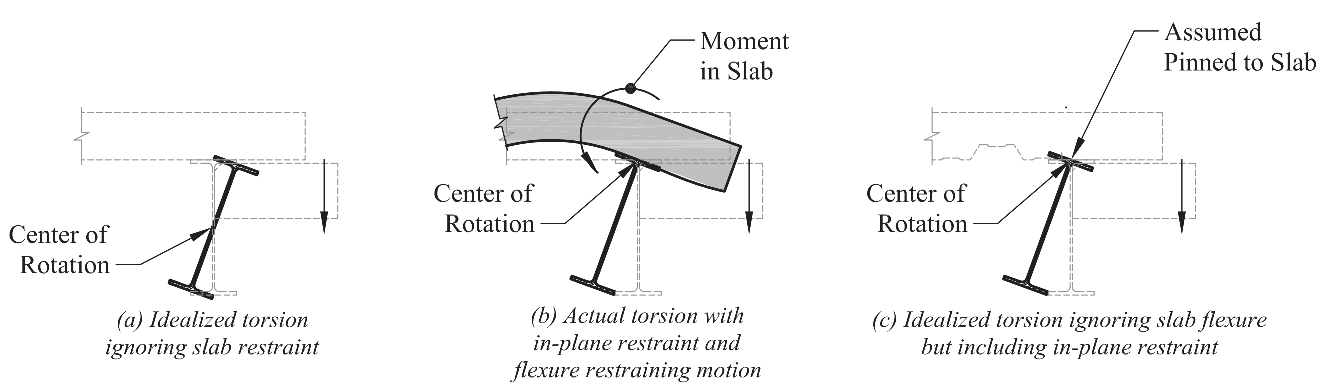


Fig. 6-3. Effects of slab flexure on torsion in spandrel beam.

condition, and some alternative details are illustrated in Figure 6-6. In each case, the framing at the opening must restrain the torsion at each end of the opening.

In the first option (Figure 6-6a), a closed cross-section is created from the wide-flange spandrel beam by adding a vertical plate connecting the tips of the top and bottom flanges over all or some portion(s) of the length between points of restraint. This will significantly increase the rotational stiffness, and preserve much of the typical spandrel beam detail. The rest of the span outside the width of the opening can continue with the typical spandrel beam section.

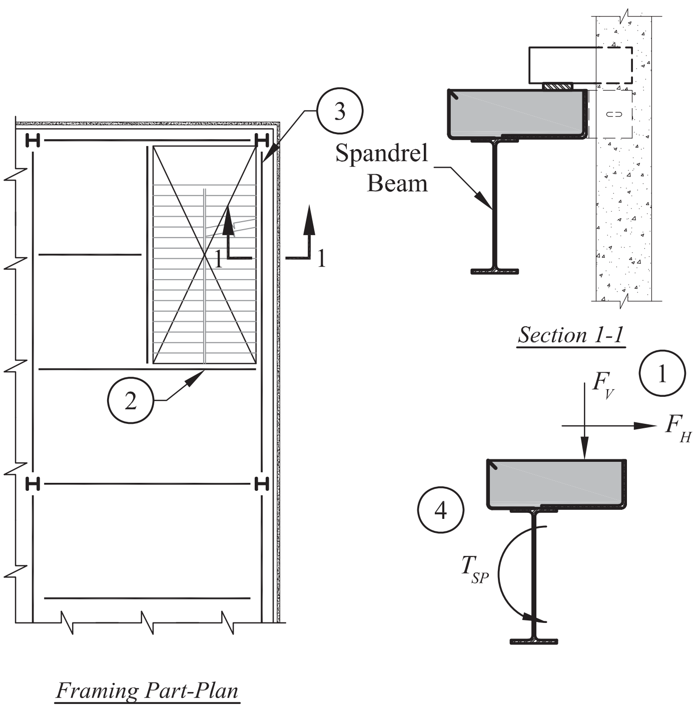
In the second option, the wide-flange shape is replaced with a hollow structural section (HSS). This also will significantly increase the rotational stiffness, but may require the designer to alter other parts of the typical spandrel beam detail, such as stiffeners, façade attachments, and connections of other framing to the spandrel beam.

In the third option, a closed shape is welded to the top (or bottom) of the wide-flange spandrel beam. This also will significantly increase the rotational stiffness, and preserve the typical spandrel beam detail, but may complicate fireproofing and the connection to the column.

(a) Flexural analogy for calculating idealized rotation ignoring slab restraint

(b) Flexural analogy for calculating rotation with top flange braced laterally but ignoring rotational restraint provided by the slab or deck

Fig. 6-4. Illustration of flexural analogy.



NOTES:

1. Eccentric façade loads along stair opening may subject the spandrel to significant torsion.
2. The floor beam can restrain torsion at the edge of the opening, with proper design and detailing.
3. The column can restrain torsion at the spandrel beam end with appropriate connections.
4. The spandrel beam must be checked for torsion and weak-axis bending, as well as for vertical loads.

Fig. 6-5. Torsion in spandrel beam at stair opening.

Examples 6.1, 6.2, 6.3, and 6.4 present the design of roof spandrel beams considering the effects of torsion due to cladding loads. Examples 6.5 and 6.6 examine the design of floor spandrel beams for cladding loads and also consider torsional effects. These examples illustrate strategies to restrain the twist in the spandrel beam and minimize the torsional stresses and rotations, thus minimizing the weight of the spandrel beam. Note that many of the torsional properties used in the examples are taken from AISC Design Guide No. 9, *Torsional Analysis of Structural Steel Members* (Seaburg and Carter, 1997).

Detailed calculations are provided for all sources of rotation and deflection that can contribute to the movement of the façade. These calculations show that not all sources of flexibility are significant, and many need not be considered in a routine design. Note, however, that it is important to consider all significant sources of flexibility.

Example 6.1—Roof Spandrel Beam with Eccentric Curtain Wall Load

For the wide-flange roof spandrel beam illustrated in Figures 6-7 and 6-8, evaluate the spandrel beam for strength and calculate the deflection for the following conditions:

Condition A. A W18×35 spandrel beam ignoring the effects of torsion.

Condition B. A W21×50 spandrel beam considering the effects of torsion, with the spandrel beam flexurally and torsionally pinned at the supporting columns.

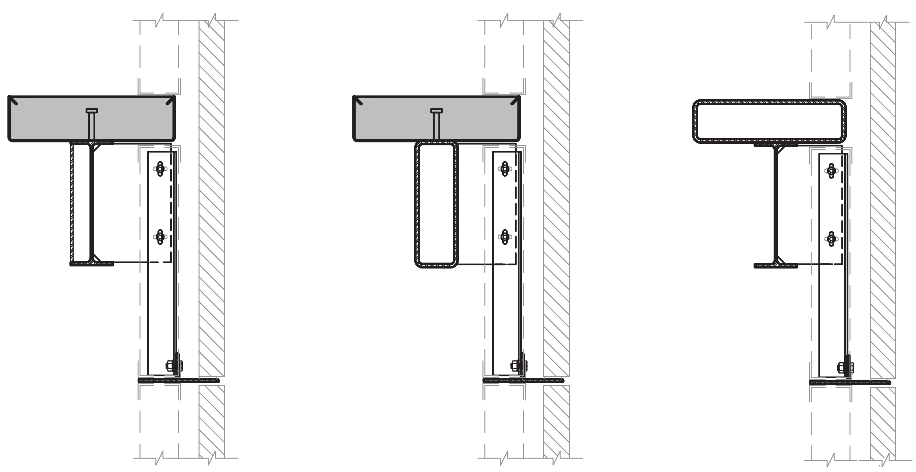
The curtain wall system requires that the maximum spandrel beam vertical deflection must be limited to 0.90 in. at the center of the span. (This limit corresponds to span/400; note that the appropriate limit will vary based on the conditions and systems used.) Use ASTM A992 material for the W-shape ($F_y = 50$ ksi).

Given:

For this example, assume that the controlling strength load combination is $1.2D + 1.6S + 0.8W$ ($D + S + W$ for deflection).

The spandrel beam span, $L = 30$ ft. The story height, $h = 14$ ft. The architect has set the edge angle face at 6 in. from the centerline of the spandrel beam; thus, l_{eod} = 6 in. The roof deck height, h_d = 3 in., and it spans s_{deck} = 10 ft to the first interior beam parallel to the spandrel beam. The deck braces the beam top flange laterally and also provides in-plane resistance to the wind load.

The roof dead load, w_{dead} = 15 psf. The roof snow load, $w_{snow} = 30$ psf. The spandrel beam also supports two stories of curtain wall cladding ($w_{curt} = 15$ psf) attached with six façade attachments at spacing, s_c = 5 ft, centered along the length of the spandrel beam. These loads are illustrated in Figure 6-9. The eccentricity between the CG of the curtain wall and the centerline of the spandrel beam, e_{cw} = 9 in. Assume the curtain wall attachment width, b_{cw} = 2 in. The cladding is exposed to a "components and cladding" wind suction, p_w = 30 psf (acting away from the building). The tributary height for wind load is one story.



Sheathing and insulation not shown for clarity.

(a) Wide-flange beam with cover plate — (b) HSS — (c) Wide-flange beam/HSS combination

Fig. 6-6. Alternative details for spandrel beams subjected to significant torsion and/or weak-axis forces.

A W18×35 spandrel beam has the following properties:

d = 17.7 in.
t_w = 0.300 in.
b_f = 6.00 in.
t_f = 0.425 in.

I_x = 510 in.^{4}
Z_x = 66.5 in.^{3}

J = 0.506 in.^{4}
C_w = 1,140 in.^{6}
a = 76.1 in.
W_{no} = 25.9 in.^{2}
Q_f = 10.7 in.^{3}
Q_w = 33.2 in.^{3}

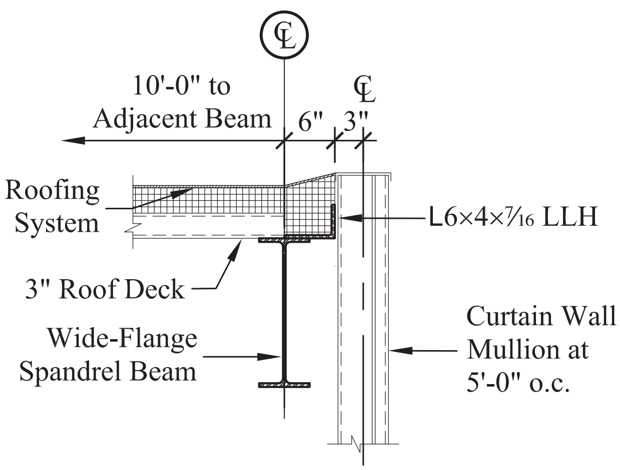


Fig. 6-7. Section of roof spandrel beam with eccentric curtain wall load.

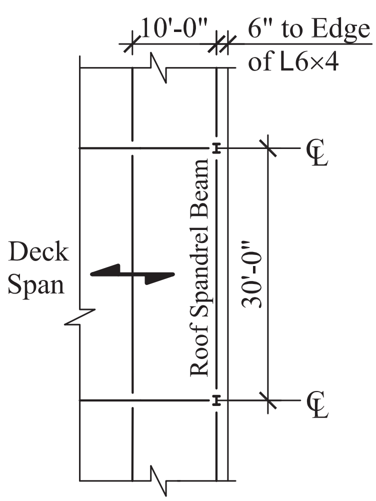


Fig. 6-8. Roof plan at spandrel beam.

A W21×50 has the following properties:

d = 20.8 in.
t_w = 0.380 in.
b_f = 6.53 in.
t_f = 0.535 in.

I_x = 984 in.^{4}
S_x = 94.5 in.^{3}
Z_x = 110 in.^{3}

J = 1.14 in.^{4}
C_w = 2,570 in.^{6}
a = 76.4 in.
W_{no} = 33.1 in.^{2}
Q_f = 17.2 in.^{3}
Q_w = 55.0 in.^{3}

Solution:

The height of cladding suspended from the spandrel beam is,

$$\begin{aligned} h_c &= 2h \\ &= 2(14\text{ ft}) \\ &= 28\text{ ft} \end{aligned}$$

The curtain wall height tributary to the spandrel beam for wind load is,

$$\begin{aligned} h_s &= \frac{h}{2} \\ &= \frac{14\text{ ft}}{2} \\ &= 7\text{ ft} \end{aligned}$$

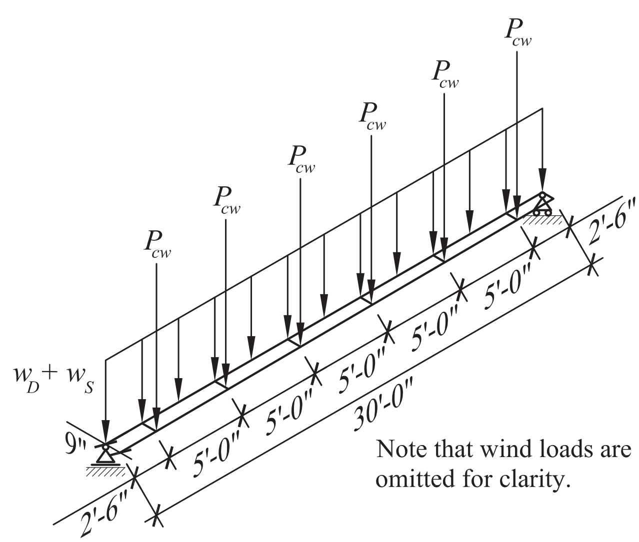


Fig. 6-9. Loads on roof spandrel beam.

The roof width tributary to the spandrel beam for dead and snow loads is,

$$
\begin{aligned}
t_r &= \frac{s_{deck}}{2} + l_{eod} \\
&= \frac{10 \text{ ft}}{2} + \frac{6 \text{ in.}}{12 \text{ in./ft}} \\
&= 5.50 \text{ ft}
\end{aligned}
$$

The uniformly distributed dead load on the spandrel beam is,

$$
\begin{aligned}
w_D &= w_{beam} + w_{dead} t_r \\
&= 0.035 \text{ kip/ft} + 0.015 \text{ kip/ft}^2 (5.50 \text{ ft}) \\
&= 0.118 \text{ kip/ft}
\end{aligned}
$$

The uniformly distributed snow load on the spandrel beam is,

$$
\begin{aligned}
w_S &= w_{snow} t_r \\
&= 0.030 \text{ kip/ft}^2 (5.50 \text{ ft}) \\
&= 0.165 \text{ kip/ft}
\end{aligned}
$$

Each curtain wall point load on the beam is,

$$
\begin{aligned}
P_{cw} &= w_{curt} h_c s_c \\
&= 0.015 \text{ kip/ft}^2 (28 \text{ ft})(5 \text{ ft}) \\
&= 2.10 \text{ kips}
\end{aligned}
$$

These loads are illustrated in Figure 6-9. The factored loads and forces are,

$$
\begin{aligned}
w_u &= 1.2 w_D + 1.6 w_S \\
&= 1.2(0.118 \text{ kip/ft}) + 1.6(0.165 \text{ kip/ft}) \\
&= 0.406 \text{ kip/ft}
\end{aligned}
$$

$$
\begin{aligned}
P_u &= 1.2 P_{cw} \\
&= 1.2(2.10 \text{ kips}) \\
&= 2.52 \text{ kips}
\end{aligned}
$$

The required flexural strength is,

$$
\begin{aligned}
M_u &= \frac{w_u L^2}{8} + P_u \left(\frac{s_c}{2} + \frac{3 s_c}{2} + \frac{5 s_c}{2} \right) \\
&= \frac{(0.406 \text{ kip/ft})(30 \text{ ft})^2}{8} \\
&\quad + 2.52 \text{ kips} \left(\frac{5 \text{ ft}}{2} + \frac{3(5 \text{ ft})}{2} + \frac{5(5 \text{ ft})}{2} \right) \\
&= 102 \text{ kip-ft}
\end{aligned}
$$

The beam is braced and, as indicated in the AISC *Steel Construction Manual* (AISC, 2005c), hereafter referred to as the AISC Manual, Table 1-1 (and beam-design tables in Part 3), the cross-section is compact. From AISC Specification Section F2, the available flexural strength is,

$$
\begin{aligned}
\phi_b M_n &= \phi_b F_y Z_x \\
&= \frac{0.90(50 \text{ ksi})(66.5 \text{ in.}^3)}{12 \text{ in./ft}} \\
&= 249 \text{ kip-ft} \geq M_u \quad \textbf{o.k.}
\end{aligned}
$$

The required shear strength is,

$$
\begin{aligned}
V_u &= \frac{w_u L}{2} + 3 P_u \\
&= \frac{0.406 \text{ kip/ft}(30 \text{ ft})}{2} + 3(2.52 \text{ kips}) \\
&= 13.7 \text{ kips}
\end{aligned}
$$

From AISC Specification Sections G1 and G2.1 (and using $\phi_v = 1.0$ and $C_v = 1.0$ because this W-shape has $h/t_w \leq 2.24\sqrt{E/F_y}$), the available shear strength is,

$$
\begin{aligned}
\phi_v V_n &= \phi_v 0.6 F_y d t_w C_v \\
&= 1.00(0.6)(50 \text{ ksi})(17.7 \text{ in.})(0.300 \text{ in.})(1.0) \\
&= 159 \text{ kips} > V_u \quad \textbf{o.k.}
\end{aligned}
$$

The total deflection includes both the deflection of the L6×4×7⁄16 edge angle and the deflection of the spandrel beam. (As noted in Tables 7-1 through 7-5, a 7⁄16-in. angle thickness may not be readily available; availability can be determined by discussion with the steel fabricator.) For the angle deflection (see Figure 6-10), each wind point load on the edge angle is,

$$
\begin{aligned}
P_{wind} &= p_w s_c \left(\frac{h}{2} \right) \\
&= 0.030 \text{ kips/ft}^2 (5 \text{ ft}) \left(\frac{14 \text{ ft}}{2} \right) \\
&= 1.05 \text{ kips}
\end{aligned}
$$

The wind point load acts at the center of the vertical angle leg; thus, $e_v = 2$ in. The moment at the base of the vertical leg of the edge angle due to the wind point load is,

$$
\begin{aligned}
M_{wa} &= P_{wind} e_v \\
&= 1.05 \text{ kips}(2 \text{ in.}) \\
&= 2.10 \text{ kip-in.}
\end{aligned}
$$

The effective width, b_{eff}, of the edge angle that resists the effects of the wind point load is determined assuming that the load spreads at a 45° angle in both directions from the curtain wall attachment width, which was given as $b_{cw} = 2$ in. Because the tip of the angle leg is at the beam flange centerline, the distance from the heel of the angle to the tip of the beam flange, $l_{oh} = 3$ in. Thus,

$$\begin{aligned} b_{eff} &= 2\tan\left(45^\circ\right)\ l_{oh} + e_v\ \ + b_{cw} \\ &= 2\tan\left(45^\circ\right)\ 3\text{ in.} + 2\text{ in.}\ \ + 2\text{ in.} \\ &= 12.0\text{ in.} \end{aligned}$$

The edge angle moment of inertia is,

$$\begin{aligned} I &= \frac{b_{eff}t^3}{12} \\ &= \frac{12.0\text{ in.}\left(\tfrac{7}{16}\text{ in.}\right)^3}{12} \\ &= 0.0837\text{ in.}^4 \end{aligned}$$

The total vertical deflection of the angle tip (using AISC Manual Table 3-23, case 22, and an additional term to account for the effect of moment M_{wa}), is

$$\begin{aligned} \Delta_{va} &= \frac{P_{cw}l_{oh}^3}{3EI} + \frac{M_{wa}l_{oh}^2}{2EI} \\ &= \frac{2.10\text{ kips }(3\text{ in.})^3}{3\left(29{,}000\text{ ksi}\right)\left(0.0837\text{ in.}^4\right)} \\ &\quad + \frac{2.10\text{ kip-in }(3\text{ in.})^2}{2\left(29{,}000\text{ ksi}\right)\left(0.0837\text{ in.}^4\right)} \\ &= 0.0117\text{ in.} \end{aligned}$$

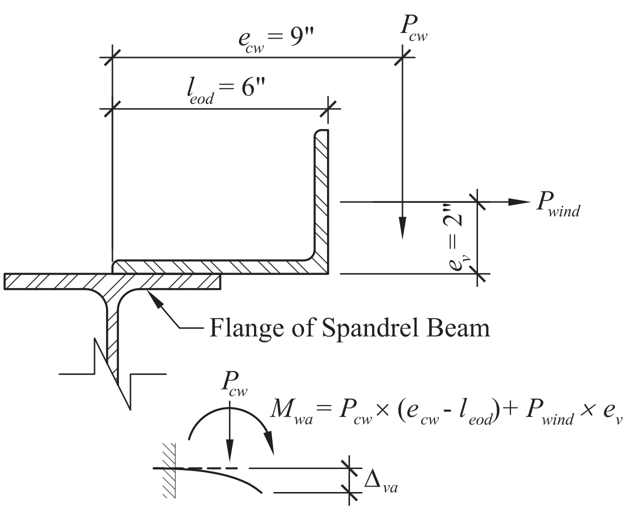


Fig. 6-10. Loads on curtain wall support angle.

The angle tip deflection is very small and often ignored in the total deflection calculation.

For the spandrel beam deflection, the uniform service load is,

$$\begin{aligned} w &= w_D + w_S \\ &= 0.118\text{ kip/ft} + 0.165\text{ kip/ft} \\ &= 0.283\text{ kip/ft} \end{aligned}$$

The corresponding deflection at midspan is,

$$\begin{aligned} \Delta_w &= \frac{5wL^4}{384EI_x} \\ &= \frac{5\left(0.283\text{ kip/ft}\right)\left(30\text{ ft}\right)^4\left(12\text{ in./ft}\right)^3}{384\left(29{,}000\text{ ksi}\right)\left(510\text{ in.}^4\right)} \\ &= 0.349\text{ in.} \end{aligned}$$

Using AISC Manual Table 3-23, case 9, the midspan deflections due to the curtain wall point loads along the beam span are as follows. For the pair of loads at 2½ ft from the beam end ($a_1 = 2\tfrac{1}{2}$ ft),

$$\begin{aligned} \Delta_{p1} &= \frac{P_{cw}a_1}{24EI_x}\left(3L^2 - 4a_1^2\right) \\ &= \frac{2.10\text{ kips}\left(2\tfrac{1}{2}\text{ ft}\right)\left(12\text{ in./ft}\right)^3}{24\left(29{,}000\text{ ksi}\right)\left(510\text{ in.}^4\right)}\left[3\left(30\text{ ft}\right)^2 - 4\left(2\tfrac{1}{2}\text{ ft}\right)^2\right] \\ &= 0.0684\text{ in.} \end{aligned}$$

For the pair of loads at 7½ ft from the beam end ($a_2 = 7\tfrac{1}{2}$ ft),

$$\begin{aligned} \Delta_{p2} &= \frac{P_{cw}a_2}{24EI_x}\left(3L^2 - 4a_2^2\right) \\ &= \frac{2.10\text{ kips}\left(7\tfrac{1}{2}\text{ ft}\right)\left(12\text{ in./ft}\right)^3}{24\left(29{,}000\text{ ksi}\right)\left(510\text{ in.}^4\right)}\left[3\left(30\text{ ft}\right)^2 - 4\left(7\tfrac{1}{2}\text{ ft}\right)^2\right] \\ &= 0.190\text{ in.} \end{aligned}$$

For the pair of loads at 12½ ft from the beam end ($a_3 = 12\tfrac{1}{2}$ ft),

$$\begin{aligned} \Delta_{p3} &= \frac{P_{cw}a_3}{24EI_x}\left(3L^2 - 4a_3^2\right) \\ &= \frac{2.10\text{ kips}\left(12\tfrac{1}{2}\text{ ft}\right)\left(12\text{ in./ft}\right)^3}{24\left(29{,}000\text{ ksi}\right)\left(510\text{ in.}^4\right)}\left[3\left(30\text{ ft}\right)^2 - 4\left(12\tfrac{1}{2}\text{ ft}\right)^2\right] \\ &= 0.265\text{ in.} \end{aligned}$$

The total vertical deflection at midspan, including the contributions of the angle and beam, is,

$$\begin{aligned}\Delta &= \Delta_{va} + \Delta_w \\ &\quad + \Delta_{p1} + \Delta_{p2} + \Delta_{p3} \\ &= 0.0117 \text{ in.} + 0.349 \text{ in.} \\ &\quad + 0.0684 \text{ in.} + 0.190 \text{ in.} + 0.265 \text{ in.} \\ &= 0.884 \text{ in.} \le 0.90 \text{ in.} \quad \textbf{o.k.}\end{aligned}$$

If torsion can be ignored, a W18×35 spandrel beam is acceptable. However, note the conclusions that are drawn when torsion is considered in Condition B.

For Condition B with a W21×50 spandrel beam, the torsional analysis method presented in AISC Design Guide No. 9, *Torsional Analysis of Structural Steel Members* (Seaburg and Carter, 1997), will be used. The rotation predicted (which is about the centroid of the beam) conservatively will be applied about the top flange, because the in-plane stiffness of the deck prevents the beam from rotating about its centroid. See also the alternative means of calculating the torsional rotation in the comments at the end of this example.

For simplicity of calculation, the torsion from the curtain wall and wind loads will be converted into an equivalent uniform torsional moment along the length of the spandrel beam, as illustrated in Figure 6-11. As described in the notes that follow, conservatively assume that the roof deck restrains the lateral translation of the top flange of the beam but that the beam's rotation is the same as if the top flange were not laterally restrained.

The uniformly distributed torsional moment on the beam due to the curtain wall is,

$$\begin{aligned}t_{cw} &= w_{curt} h_c e_{cw} \\ &= 0.015 \text{ kip/ft}^2 (28 \text{ ft}) \left(\frac{9 \text{ in.}}{12 \text{ in./ft}} \right) \\ &= 0.315 \text{ kip-ft/ft}\end{aligned}$$

The uniformly distributed torsional moment on the beam due to wind is,

$$\begin{aligned}t_{wind} &= p_w \left(\frac{h}{2} \right) e_v \\ &= 0.030 \text{ kip/ft}^2 \left(\frac{14 \text{ ft}}{2} \right) \left(\frac{2 \text{ in.}}{12 \text{ in/ft}} \right) \\ &= 0.0350 \text{ kip-ft/ft}\end{aligned}$$

The required strength for torsional effects is,

$$\begin{aligned}t_u &= 1.2 t_{cw} + 0.8 t_{wind} \\ &= 1.2(0.315 \text{ kip-ft/ft}) + 0.8(0.0350 \text{ kip-ft/ft}) \\ &= 0.406 \text{ kip-ft/ft}\end{aligned}$$

The uniformly distributed dead load on the spandrel beam is,

$$\begin{aligned}w_D &= w_{beam} + w_{dead} t_r \\ &= 0.050 \text{ kip/ft} + 0.015 \text{ kip/ft}^2 (5.50 \text{ ft}) \\ &= 0.133 \text{ kip/ft}\end{aligned}$$

and the uniformly distributed snow load on the spandrel beam, w_s = 0.165 kip/ft, as calculated previously. The factored loads and forces are,

$$\begin{aligned}w_u &= 1.2 w_D + 1.6 w_S \\ &= 1.2(0.133 \text{ kip/ft}) + 1.6(0.165 \text{ kip/ft}) \\ &= 0.424 \text{ kip/ft}\end{aligned}$$

and P_u = 2.52 kips, t_{cw} = 0.315 kip-ft/ft, t_{wind} = 0.0350 kip-ft/ft, and t_u = 0.406 kip-ft/ft, as calculated previously. From AISC Design Guide No. 9 Appendix B, case 4, with

$$\begin{aligned}\frac{L}{a} &= \frac{30 \text{ ft} \times 12 \text{ in/ft}}{76.4 \text{ in.}} \\ &= 4.71\end{aligned}$$

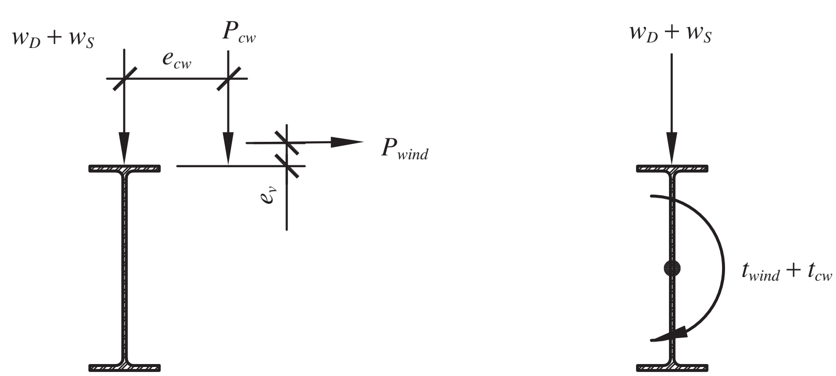


Fig. 6-11. Vertical and torsional loads on spandrel beam.

the torsional functions at midspan are,

$$\theta_u\left(\frac{GJ}{t_u}\right)\left(\frac{1}{2a^2}\right) = 0.97 \text{ rad.}$$

$$\theta_u''\left(\frac{GJ}{t_u}\right) = 0.81 \text{ rad.}$$

and the torsional function at the supports is,

$$\theta_u'\left(\frac{GJ}{t_u}\right)\left(\frac{L}{10a^2}\right) = 0.62 \text{ rad.}$$

θ''' is not applicable because warping does not affect the shear stresses in the web.

For the interaction of flexure and torsion at midspan,

$$\begin{aligned}\theta_u'' &= 0.81 \text{ rad.}\frac{t_u}{GJ}\\ &= 0.81 \text{ rad.}\frac{(0.406 \text{ kip-ft/ft})}{(11{,}200 \text{ ksi})(1.14 \text{ in.}^4)}\\ &= 2.58\times10^{-5} \text{ rad./in.}^2\end{aligned}$$

$$\begin{aligned}\sigma_{wsu} &= EW_{no}\theta_u''\\ &= (29{,}000 \text{ ksi})(33.1 \text{ in.}^2)(2.58\times10^{-5} \text{ rad./in.}^2)\\ &= 24.8 \text{ ksi}\end{aligned}$$

$$\begin{aligned}M_u &= \frac{w_u L^2}{8} + P_u\left(\frac{s_c}{2}+\frac{3s_c}{2}+\frac{5s_c}{2}\right)\\ &= \frac{(0.424 \text{ kip/ft})(30 \text{ ft})^2}{8}\\ &\quad + 2.52 \text{ kips}\left(\frac{5 \text{ ft}}{2}+\frac{3(5 \text{ ft})}{2}+\frac{5(5 \text{ ft})}{2}\right)\\ &= 104 \text{ kip-ft}\end{aligned}$$

AISC Design Guide No. 9 Equation 4.16a is based on the summation of elastic stresses; therefore, the design flexural strength, $\phi_b M_n$, is,

$$\begin{aligned}\phi_b M_n &= \phi_b F_y S_x\\ &= \frac{0.90(50 \text{ ksi})(94.5 \text{ in.}^3)}{12 \text{ in./ft}}\\ &= 354 \text{ kip-ft}\end{aligned}$$

Using AISC Design Guide No. 9 Equation 4-16a, the interaction of combined normal stresses assuming the predominant limit state is yielding is,

$$\begin{aligned}\frac{M_u}{\phi_b F_y S_x} + \frac{\sigma_{wsu}}{0.9F_y} &= \frac{104 \text{ kip-ft}}{354 \text{ kip-ft}} + \frac{24.8 \text{ ksi}}{0.9(50 \text{ ksi})}\\ &= 0.845 \le 1.0 \quad \textbf{o.k.}\end{aligned}$$

For the interaction of shear and torsion at the support,

$$\begin{aligned}\theta' &= 0.62 \text{ rad.}\left(\frac{t_u}{GJ}\right)\left(\frac{10a^2}{L}\right)\\ &= 0.62 \text{ rad.}\frac{(0.406 \text{ kip-ft/ft})}{(11{,}200 \text{ ksi})(1.14 \text{ in.}^4)}\frac{10(76.4 \text{ in.})^2}{30 \text{ ft}(12 \text{ in./ft})}\\ &= 0.00320 \text{ rad./in.}\end{aligned}$$

$$\begin{aligned}\tau_t &= Gt_w\theta'\\ &= 11{,}200 \text{ ksi}(0.380 \text{ in.})(0.00320 \text{ rad./in.})\\ &= 13.6 \text{ ksi}\end{aligned}$$

$$\begin{aligned}V_u &= \frac{w_u L}{2} + 3P_u\\ &= \frac{0.424 \text{ kip/ft(30 ft)}}{2} + 3(2.52 \text{ kips})\\ &= 13.9 \text{ kips}\end{aligned}$$

From AISC *Specification* Sections G1 and G2.1 (and using ϕ_v = 1.0 and C_v = 1.0 because this W-shape has $h/t_w \le 2.24\sqrt{E/F_y}$), the available shear strength is,

$$\begin{aligned}\phi_v V_n &= \phi_v 0.6F_y dt_w C_v\\ &= 1.00(0.6)(50 \text{ ksi})(20.8 \text{ in.})(0.380 \text{ in.})(1.0)\\ &= 237 \text{ kips}\end{aligned}$$

The interaction of pure torsion shear stress and direct shear is,

$$\begin{aligned}\frac{V_u}{\phi_v V_n} + \frac{\tau_t}{0.9(0.6F_y)} &= \frac{13.9 \text{ kips}}{237 \text{ kips}} + \frac{13.6 \text{ ksi}}{0.9(0.6)(50 \text{ ksi})}\\ &= 0.562 \le 1.0 \quad \textbf{o.k.}\end{aligned}$$

The total vertical deflection of the angle tip, Δ_{va} = 0.0117 in. as calculated previously. For the spandrel beam deflection, the uniform service load is,

$$\begin{aligned}w &= w_D + w_S\\ &= 0.133 \text{ kip/ft} + 0.165 \text{ kip/ft}\\ &= 0.298 \text{ kip/ft}\end{aligned}$$

The corresponding deflection at midspan is,

$$\begin{aligned}\Delta_w &= \frac{5wL^4}{384EI_x}\\ &= \frac{5(0.298 \text{ kip/ft})(30 \text{ ft})^4(12 \text{ in./ft})^3}{384(29{,}000 \text{ ksi})(984 \text{ in.}^4)}\\ &= 0.190 \text{ in.}\end{aligned}$$

Using AISC *Manual* Table 3-23, case 9, the midspan deflections due to the curtain wall point loads along the beam span are as follows. For the pair of loads at 2½ ft from the beam end (a_1 = 2½ ft),

$$\begin{aligned}\Delta_{p1} &= \frac{P_{cw}a_1}{24EI_x}\left(3L^2 - 4a_1^2\right)\\ &= \frac{2.10 \text{ kips}(2\tfrac{1}{2} \text{ ft})(12 \text{ in./ft})^3}{24(29{,}000 \text{ ksi})(984 \text{ in.}^4)}\left[3(30 \text{ ft})^2 - 4(2\tfrac{1}{2} \text{ ft})^2\right]\\ &= 0.0354 \text{ in.}\end{aligned}$$

For the pair of loads at 7½ ft from the beam end (a_2 = 7½ ft),

$$\begin{aligned}\Delta_{p2} &= \frac{P_{cw}a_2}{24EI_x}\left(3L^2 - 4a_2^2\right)\\ &= \frac{2.10 \text{ kips}(7\tfrac{1}{2} \text{ ft})(12 \text{ in./ft})^3}{24(29{,}000 \text{ ksi})(984 \text{ in.}^4)}\left[3(30 \text{ ft})^2 - 4(7\tfrac{1}{2} \text{ ft})^2\right]\\ &= 0.0984 \text{ in.}\end{aligned}$$

For the pair of loads at 12½ ft from the beam end (a_3 = 12½ ft),

$$\begin{aligned}\Delta_{p3} &= \frac{P_{cw}a_3}{24EI_x}\left(3L^2 - 4a_3^2\right)\\ &= \frac{2.10 \text{ kips}(12\tfrac{1}{2} \text{ ft})(12 \text{ in./ft})^3}{24(29{,}000 \text{ ksi})(984 \text{ in.}^4)}\left[3(30 \text{ ft})^2 - 4(12\tfrac{1}{2} \text{ ft})^2\right]\\ &= 0.137 \text{ in.}\end{aligned}$$

For the rotational components of the deflection calculations,

$$\begin{aligned}t &= t_{cw} + t_{wind}\\ &= 0.315 \text{ kip-ft/ft} + 0.0350 \text{ kip-ft/ft}\\ &= 0.350 \text{ kip-ft/ft}\end{aligned}$$

The rotation of the beam at midspan is based on the service level torsion,

$$\begin{aligned}\theta &= 0.97 \text{ rad.}\left(\frac{t}{GJ}\right)2a^2\\ &= 0.97 \text{ rad.}\frac{(0.350 \text{ kip-ft/ft})}{(11{,}200 \text{ ksi})(1.14 \text{ in.}^4)}(2)(76.4 \text{ in.})^2\\ &= 0.310 \text{ rad. (or } 17.8°)\end{aligned}$$

As illustrated in Figure 6-12, the heel of the angle is at the same vertical elevation as, and 6 in. to the right of, the point of rotation, which is set at the center of the top flange. With a clockwise rotation of 17.8°, the heel of the angle will deflect vertically by,

$$\begin{aligned}\Delta_{vrb} &= l_{eod}\sin(17.8°)\\ &= (6.00 \text{ in.})\sin(17.8°)\\ &= 1.83 \text{ in.}\end{aligned}$$

The total vertical deflection at midspan, including the contributions of the angle and beam, is,

$$\begin{aligned}\Delta &= \Delta_{va} + \Delta_w\\ &\quad + \Delta_{p1} + \Delta_{p2} + \Delta_{p3}\\ &\quad + \Delta_{vrb}\\ &= 0.0117 \text{ in.} + 0.190 \text{ in.}\\ &\quad + 0.0354 \text{ in.} + 0.0984 \text{ in.} + 0.137 \text{ in.}\\ &\quad + 1.83 \text{ in.}\\ &= 2.30 \text{ in.} > 0.9 \text{ in. } \textbf{n.g.}\end{aligned}$$

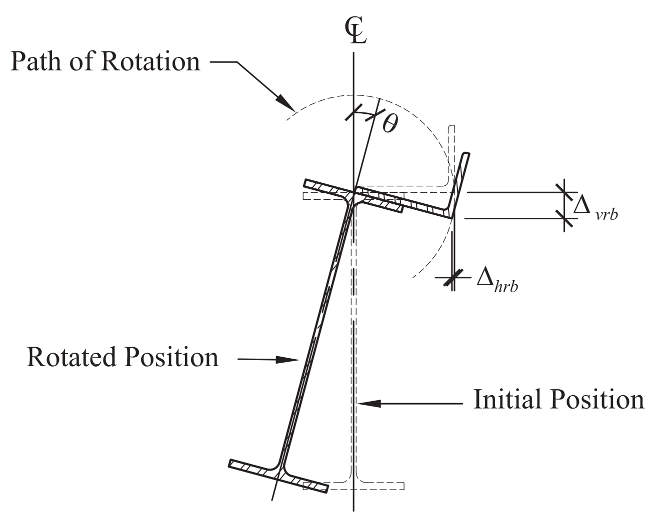


Fig. 6-12. Rigid body rotation about center of top flange.

This total vertical deflection substantially exceeds the deflection limit, primarily due to the torsional effects. In fact, the torsional effects alone for this beam exceed the stated deflection limit for the curtain wall system.

The heel of the angle will also deflect horizontally (inward) by,

$$\begin{aligned}\Delta_{hrb} &= l_{eod} - l_{eod}\cos\left(17.8^\circ\right)\\ &= 6.00\text{ in.} - \left(6.00\text{ in.}\right)\cos\left(17.8^\circ\right)\\ &= 0.287\text{ in.}\end{aligned}$$

This is an additional out-of-plane movement that must be considered. Note that the horizontal deflection of the vertical leg of the edge angle has been neglected because the fillet stiffens its vertical leg to the point where the horizontal deflection will be very small. When the vertical leg is long and the thickness small, the horizontal deflection may be non-negligible.

Thus, even a spandrel beam as heavy as a W21×50 has substantial torsional rotations. In the following examples, a system that uses roll beams to reduce the torsion on the spandrel beam will be designed.

Comments:
This example shows that an eccentric cladding load will cause significant twist in a spandrel beam that does not have adequate strength or restraint to prevent the twist. Even when eccentricities appear small it may be important to include the effects of torsion on wide-flange spandrel beams.

For simplicity, this example does not account for the increase in strong-axis moment of inertia due to the addition of the edge angle. This additional amount can be considered, when desired.

The edge angle size was given as adequate for the purposes of this problem. A complete design of this system would include evaluation of the flexural strength of the angle section to resist the curtain wall and wind loads.

As an alternative approach, the torsional effects and rotation of the beam can be calculated using a "flexural analogy." This method converts the applied torsion into a force couple acting at the top and bottom flanges of the beam. The force at the top flange is resisted by the deck. The force at the bottom flange is resisted by weak-axis bending of the bottom half of the spandrel beam. The calculated deflection is then converted into an equivalent rotation about the top flange.

The weak-axis moment of inertia of the bottom half of the W21×50 beam is,

$$\begin{aligned}I_{y\ FA} &= \frac{I_y}{2}\\ &= \frac{24.9\text{ in.}^4}{2}\\ &= 12.5\text{ in.}^4\end{aligned}$$

The equivalent uniform load applied laterally to the bottom flange to generate the torsional moment is,

$$\begin{aligned}w_{FA} &= \frac{t}{d}\\ &= \frac{0.350\text{ kip-ft/ft }(12\text{ in./ft})}{20.8\text{ in.}}\\ &= 0.202\text{ kip/ft}\end{aligned}$$

The corresponding deflection of the bottom flange is,

$$\begin{aligned}\Delta_{bf\ FA} &= \frac{5w_{FA}L^4}{384EI_{y\ FA}}\\ &= \frac{5\left(0.202\text{ kip/ft}\right)\left(30\text{ ft}\right)^4\left(12\text{ in./ft}\right)^3}{384\left(29{,}000\text{ ksi}\right)\left(12.5\text{ in.}^4\right)}\\ &= 10.2\text{ in.}\end{aligned}$$

Because the beam is forced to rotate about its top flange, the full beam depth can be used when calculating the angle of rotation, which is,

$$\begin{aligned}\theta_{FA} &= \sin^{-1}\left(\frac{\Delta_{bf\ FA}}{d}\right)\\ &= \sin^{-1}\left(\frac{10.2\text{ in.}}{20.8\text{ in.}}\right)\\ &= 29.4^\circ\end{aligned}$$

In this case, the rotation predicted by the flexural analogy is approximately 1.7 times that predicted previously using the method in AISC Design Guide No. 9. This illustrates that, while the flexural analogy requires less computational effort, it can be unrealistically conservative on long spans like this one. The following examples examine this further for shorter spans where the flexural analogy becomes more appropriate and the savings in computational effort is justified.

As another common and useful simplification for multiple point loads uniformly spaced along the length of a spandrel

beam, it is often acceptable to convert the point loads to a uniform load. For example, for deflection of the W18×35 in Condition A,

$$
\begin{aligned}
w_{eq} &= \frac{6P_{cw}}{L} \\
&= \frac{6(2.10 \text{ kips})}{30 \text{ ft}} \\
&= 0.420 \text{ kip/ft}
\end{aligned}
$$

$$
\begin{aligned}
\Delta_{w\,eq} &= \frac{5wl^4}{384EI} \\
&= \frac{5(0.420 \text{ kip/ft})(30 \text{ ft})^4 (12 \text{ in./ft})^3}{384(29{,}000 \text{ ksi})(510 \text{ in.}^4)} \\
&= 0.518 \text{ in.}
\end{aligned}
$$

This compares very favorably to the deflection calculated previously as,

$$
\begin{aligned}
\Delta &= \Delta_{p1} + \Delta_{p2} + \Delta_{p3} \\
&= 0.0684 \text{ in.} + 0.190 \text{ in.} + 0.265 \text{ in.} \\
&= 0.523 \text{ in.}
\end{aligned}
$$

Example 6.2—Roof Spandrel Beam with Eccentric Curtain Wall Load—Torsion Restrained with Roll Beams

Repeat Example 6.1 using Condition B, but with W10×12 roll beams perpendicular to a W18×40 spandrel beam at 10 ft on center (s_{RB} = 10 ft), as illustrated in Figures 6-13 and 6-14, and used to limit rotational deflections and torsional effects on the spandrel beam. Also design the connection of the roll beam to the spandrel beam.

Given:

The spandrel beam span for torsion, L_T = 10 ft. The roll beam length, L_{RB} = 10 ft. The roll beams do not carry any floor loads. The first interior beam adjacent to the spandrel beam is a W16×26. Neglect the vertical deflection of the edge angle, which was small as calculated in Example 6.1.

The roll beam is connected by bolting to a full-depth stiffener on the spandrel (see Figure 6-13). There are three ¾-in.-diameter ASTM A325-N bolts with spacing, s = 3 in. The distance from the centroid of the bolt group to the furthest bolt, s_{max} = 3 in.

A W18×40 spandrel has the following properties:

d = 17.9 in.
t_w = 0.315 in.
b_f = 6.02 in.
t_f = 0.525 in.

I_x = 612 in.^{4}
S_x = 68.4 in.^{3}
Z_x = 78.4 in.^{3}

J = 0.810 in.^{4}
C_w = 1,440 in.^{6}
a = 67.8 in.
W_{no} = 26.1 in.^{2}
Q_f = 13.3 in.^{3}
Q_w = 39.2 in.^{3}

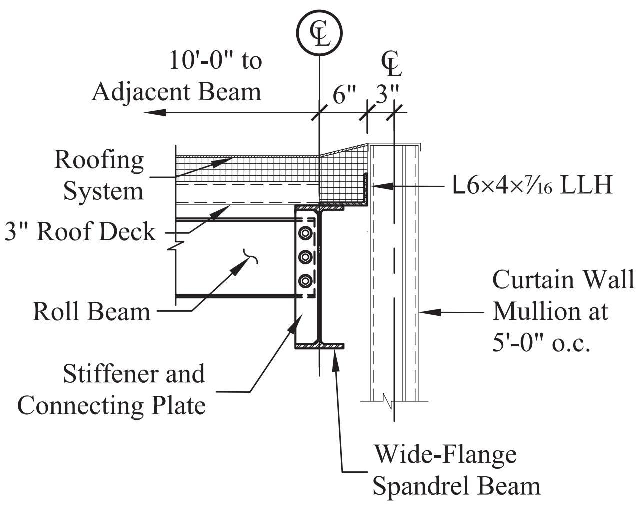


Fig. 6-13. Section of roof spandrel beam with eccentric curtain wall.

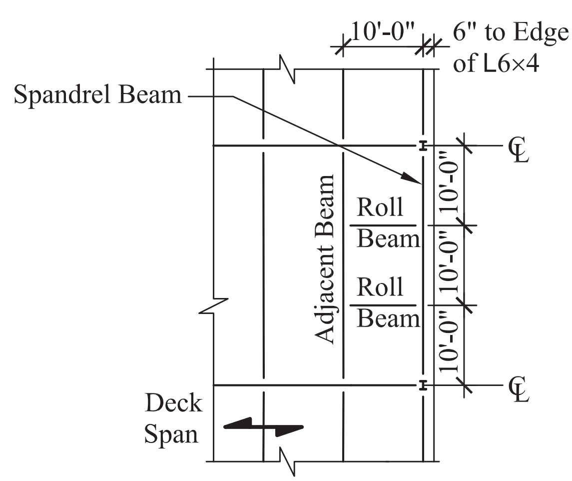


Fig. 6-14. Roof plan at spandrel beam.

The W10×12 roll beams have the following property:

$$I_{x\,RB} = 53.8 \text{ in.}^4$$

The W16×26 adjacent interior beam has the following property:

$$I_{xa} = 301 \text{ in.}^4$$

Solution:

The loading is as illustrated in Figure 6-15. For simplicity, the wind and curtain wall loads (see Figure 6-16) will be converted into equivalent uniformly distributed torsional moments on the spandrel beam. The equivalent uniformly distributed torsional moment due to the curtain wall loads on the spandrel beam is,

$$\begin{aligned} t_{cw} &= w_{curt} h_c e_{cw} \\ &= 0.015 \text{ kip/ft}^2 \left(28 \text{ ft}\right)\left(\frac{9 \text{ in.}}{12 \text{ in./ft}}\right) \\ &= 0.315 \text{ kip-ft/ft} \end{aligned}$$

The equivalent uniformly distributed torsional moment due to wind on the spandrel beam is,

$$\begin{aligned} t_{wind} &= p_w \left(\frac{h}{2}\right)\left(e_v\right) \\ &= 0.030 \text{ kip/ft}^2 \left(\frac{14 \text{ ft}}{2}\right)\left(\frac{2 \text{ in.}}{12 \text{ in./ft}}\right) \\ &= 0.0350 \text{ kip-ft/ft} \end{aligned}$$

The required strength for torsional effects is,

$$\begin{aligned} t_u &= 1.2t_{cw} + 0.8t_{wind} \\ &= 1.2\left(0.315 \text{ kip-ft/ft}\right) + 0.8\left(0.0350 \text{ kip-ft/ft}\right) \\ &= 0.406 \text{ kip-ft/ft} \end{aligned}$$

Because the roll beams restrain the torsion on the spandrel beam, they impose vertical reactions on the spandrel beam and the adjacent interior beam (see Figure 6-17). Thus, to calculate the required flexural strength of the spandrel beam, the moments in the roll beams must first be determined. Also for simplicity, the torsional continuity in the spandrel beam at the roll beams will be ignored, and conservatively it will be assumed that the spandrel beam is torsionally pinned at each roll beam. Alternatively, it is permissible to consider the beneficial effects of the continuity.

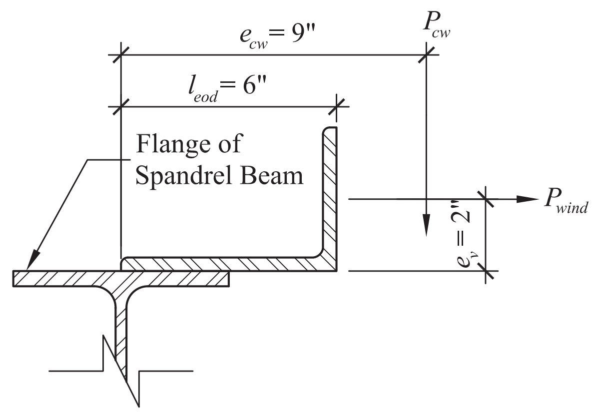


Fig. 6-16. Torsional loads on spandrel beam.

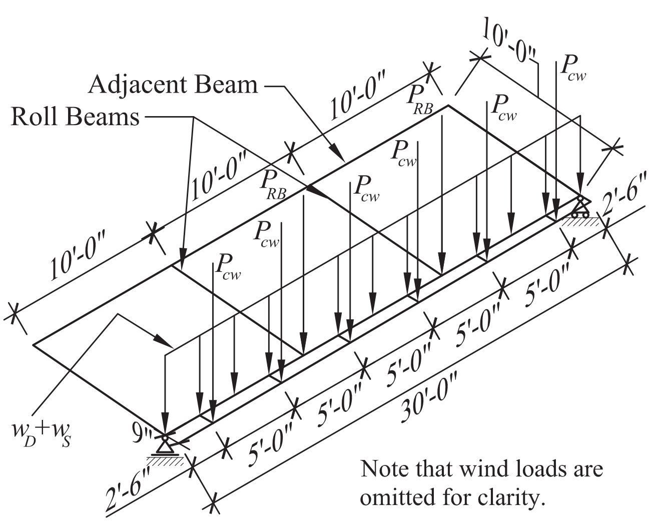


Fig. 6-15. Loads on roof spandrel beam.

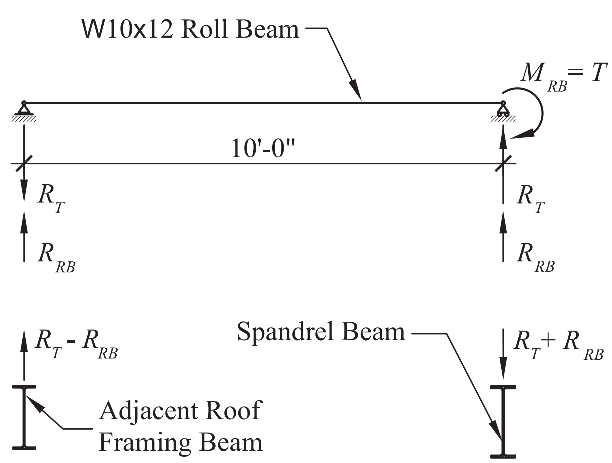


Fig. 6-17. Loads on and reactions due to roll beam.

The torsional moment in the spandrel on each side of each roll beam is,

$$
\begin{aligned}
T_u &= t_u\left(\frac{L_T}{2}\right) \\
&= \left(0.406 \text{ kip-ft/ft}\right)\left(\frac{10 \text{ ft}}{2}\right) \\
&= 2.03 \text{ kip-ft}
\end{aligned}
$$

Each roll beam provides a flexural moment that resists twice this torsional moment (T_u is present on both sides of each roll beam).

$$
\begin{aligned}
R_{Tu} &= \frac{2T_u}{L_{RB}} \\
&= \frac{2\left(2.03 \text{ kip-ft}\right)}{10 \text{ ft}} \\
&= 0.406 \text{ kip}
\end{aligned}
$$

The additional vertical reaction on the spandrel beam due to the self weight of the roll beam is,

$$
\begin{aligned}
R_{RBu} &= 1.2w_{RB}\left(\frac{L_{RB}}{2}\right) \\
&= 1.2\left(0.012 \text{ kip/ft}\right)\left(\frac{10 \text{ ft}}{2}\right) \\
&= 0.0720 \text{ kip}
\end{aligned}
$$

The total factored point load on the spandrel beam at each roll beam is,

$$
\begin{aligned}
P_{RBu} &= R_{Tu} + R_{RBu} \\
&= 0.406 \text{ kip} + 0.0720 \text{ kip} \\
&= 0.478 \text{ kip}
\end{aligned}
$$

The uniformly distributed dead load on the spandrel beam is,

$$
\begin{aligned}
w_D &= w_{beam} + w_{dead}t_r \\
&= 0.040 \text{ kip/ft} + 0.015 \text{ kip/ft}^2\left(5.50 \text{ ft}\right) \\
&= 0.123 \text{ kip/ft}
\end{aligned}
$$

and the uniformly distributed snow load on the spandrel beam, w_s = 0.165 kip/ft, as calculated previously. The factored load is,

$$
\begin{aligned}
w_u &= 1.2w_D + 1.6w_s \\
&= 1.2\left(0.123 \text{ kip/ft}\right) + 1.6\left(0.165 \text{ kip/ft}\right) \\
&= 0.412 \text{ kip/ft}
\end{aligned}
$$

As calculated previously, P_u = 2.52 kips. From AISC Design Guide No. 9 Appendix B, case 4 with,

$$
\begin{aligned}
\frac{L_T}{a} &= \frac{10 \text{ ft}\left(12 \text{ in./ft}\right)}{67.8 \text{ in.}} \\
&= 1.77
\end{aligned}
$$

The torsional functions at the middle of the torsional spans (span between roll beams) are,

$$
\theta_u\left(\frac{GJ}{t_u}\right)\left(\frac{1}{2a^2}\right) = 0.037 \text{ rad.}
$$

$$
\theta_u''\left(\frac{GJ}{t_u}\right) = 0.24 \text{ rad.}
$$

The torsional function at the ends of the torsional spans (at the roll beams) is,

$$
\theta_u'\left(\frac{GJ}{t_u}\right)\left(\frac{L}{10a^2}\right) \approx 0.04 \text{ rad.}
$$

θ''' is not applicable because warping does not affect the shear stresses in the web.

For the interaction of flexure and torsion at midspan,

$$
\begin{aligned}
\theta_u'' &= 0.24 \text{ rad.}\left(\frac{t_u}{GJ}\right) \\
&= 0.24 \text{ rad.}\frac{\left(0.406 \text{ kip-ft/ft}\right)}{\left(11{,}200 \text{ ksi}\right)\left(0.810 \text{ in.}^4\right)} \\
&= 1.07\times10^{-5} \text{ rad./in.}^2
\end{aligned}
$$

$$
\begin{aligned}
\sigma_{wsu} &= EW_{no}\theta_u'' \\
&= \left(29{,}000 \text{ ksi}\right)\left(26.1 \text{ in.}^2\right)\left(1.07\times10^{-5} \text{ rad./in.}^2\right) \\
&= 8.10 \text{ ksi}
\end{aligned}
$$

$$
\begin{aligned}
M_u &= \frac{w_uL^2}{8} + P_u\left(\frac{s_c}{2}+\frac{3s_c}{2}+\frac{5s_c}{2}\right) + P_{RBu}s_{RB} \\
&= \frac{0.412 \text{ kip/ft}\left(30 \text{ ft}\right)^2}{8} \\
&\quad + 2.52 \text{ kips}\left(\frac{5 \text{ ft}}{2}+\frac{3\left(5 \text{ ft}\right)}{2}+\frac{5\left(5 \text{ ft}\right)}{2}\right) \\
&\quad + 0.478 \text{ kip}\left(10 \text{ ft}\right) \\
&= 109 \text{ kip-ft}
\end{aligned}
$$

$$
\begin{aligned}
\phi_bF_yS_x &= \frac{0.90\left(50 \text{ ksi}\right)\left(68.4 \text{ in.}^3\right)}{12 \text{ in./ft}} \\
&= 257 \text{ kip-ft}
\end{aligned}
$$

Using AISC Design Guide No. 9 Equation 4-16a, the interaction of combined normal stresses assuming the predominant limit state is yielding is,

$$\frac{M_u}{\phi_b M_n} \quad \frac{\sigma_{wsu}}{0.9F_y} \quad \frac{109 \text{ kip-ft}}{257 \text{ kip-ft}} + \frac{8.10 \text{ ksi}}{0.9(50 \text{ ksi})}$$
$$= 0.604 \leq 1.0 \quad \textbf{o.k.}$$

For the interaction of shear and torsion at the support,

$$\theta' = 0.04 \text{ rad.} \left(\frac{t_u}{GJ}\right)\left(\frac{10a^2}{L}\right)$$
$$= 0.04 \text{ rad.} \frac{(0.406 \text{ kip-ft/ft})}{(11{,}200 \text{ ksi})(0.810 \text{ in.}^4)} \frac{10(67.8 \text{ in.})^2}{30 \text{ ft}(12 \text{ in./ft})}$$
$$= 0.000229 \text{ rad./in.}$$

$$\tau_t = Gt_w\theta'$$
$$= 11{,}200 \text{ ksi}(0.380 \text{ in.})(0.000229 \text{ rad./in.})$$
$$= 0.975 \text{ ksi}$$

θ''' is not applicable because warping does not affect the shear stresses in the web.

$$V_u = \frac{w_u L}{2} + 3P_u + P_{RBu}$$
$$= \frac{0.412 \text{ kip/ft}(30 \text{ ft})}{2} + 3(2.52 \text{ kips}) + 0.478 \text{ kip}$$
$$= 14.2 \text{ kips}$$

From AISC *Specification* Sections G1 and G2.1 (and using ϕ_v = 1.0 and C_v = 1.0 because this W-shape has $h/t_w \leq 2.24\sqrt{E/F_y}$), the available shear strength is,

$$\phi_v V_n = \phi_v 0.6F_y dt_w C_v$$
$$= 1.00(0.6)(50 \text{ ksi})(20.8 \text{ in.})(0.380 \text{ in.})(1.0)$$
$$= 237 \text{ kips}$$

The interaction of pure torsion shear stress and direct shear is,

$$\frac{V_u}{\phi_v V_n} + \frac{\tau_t}{0.9(0.6F_y)} = \frac{14.2 \text{ kips}}{237 \text{ kips}} + \frac{0.975 \text{ ksi}}{0.9(0.6)(50 \text{ ksi})}$$
$$= 0.0960 \leq 1.0 \quad \textbf{o.k.}$$

As calculated previously, w_D = 0.123 kip/ft, w_S = 0.165 kip/ft, and P_{cw} = 2.10 kips.

The service torsion on the spandrel beam is,

$$t = t_{cw} + t_{wind}$$
$$= 0.315 \text{ kip-ft/ft} + 0.0350 \text{ kip-ft/ft}$$
$$= 0.350 \text{ kip-ft/ft}$$

The service torsional moment on each roll beam is,

$$T = t\left(\frac{L_T}{2}\right)$$
$$= 0.350 \text{ kip-ft/ft}\left(\frac{10 \text{ ft}}{2}\right)$$
$$= 1.75 \text{ kip-ft}$$

The service vertical reaction on the spandrel due to the end moment on the roll beam is,

$$R_T = \frac{2T}{L_{RB}}$$
$$= \frac{2(1.75 \text{ kip-ft})}{10 \text{ ft}}$$
$$= 0.350 \text{ kip}$$

The service vertical reaction on the spandrel due to the self weight of the roll beam is,

$$R_{RB} = w_{RB}\frac{L_{RB}}{2}$$
$$= 0.012 \text{ kip/ft}\left(\frac{10 \text{ ft}}{2}\right)$$
$$= 0.0600 \text{ kip}$$

The service point load on the spandrel at each roll beam is,

$$P_{RB} = R_T + R_{RB}$$
$$= 0.350 \text{ kip} + 0.0600 \text{ kip}$$
$$= 0.410 \text{ kip}$$

The uniform service load is,

$$w = w_D + w_S$$
$$= 0.123 \text{ lb/ft} + 0.165 \text{ lb/ft}$$
$$= 0.288 \text{ kip/ft}$$

The corresponding deflection at midspan is,

$$\Delta_w = \frac{5wl^4}{384EI_x}$$
$$= \frac{5(0.288 \text{ kip/ft})(30 \text{ ft})^4 (12 \text{ in./ft})^3}{384(29{,}000 \text{ ksi})(612 \text{ in.}^4)}$$
$$= 0.296 \text{ in.}$$

Using AISC Manual Table 3-23, case 9, the midspan deflections due to the curtain wall point loads along the beam span are as follows. For the pair of loads at 2½ ft from the beam end (a_1 = 2½ ft),

$$\Delta_{p1} = \frac{P_{cw}a_1}{12EI}\left(\frac{3}{4}L^2 - a_1^2\right)$$
$$= \frac{2.10\text{ kips}\left(2½\text{ ft}\right)\left(12\text{ in./ft}\right)^3}{12\left(29{,}000\text{ ksi}\right)\left(612\text{ in.}^4\right)}\left[\frac{3}{4}\left(30\text{ ft}\right)^2 - \left(2½\text{ ft}\right)^2\right]$$
$$= 0.0285\text{ in.}$$

For the pair of loads at 7½ ft from the beam end (a_2 = 7½ ft),

$$\Delta_{p2} = \frac{P_{cw}a_2}{12EI}\left(\frac{3}{4}L^2 - a_2^2\right)$$
$$= \frac{2.10\text{ kips}\left(7½\text{ ft}\right)\left(12\text{ in./ft}\right)^3}{12\left(29{,}000\text{ ksi}\right)\left(612\text{ in.}^4\right)}\left[\frac{3}{4}\left(30\text{ ft}\right)^2 - \left(7½\text{ ft}\right)^2\right]$$
$$= 0.0791\text{ in.}$$

For the pair of loads at 12½ ft from the beam end (a_3 = 12½ ft),

$$\Delta_{p3} = \frac{P_{cw}a_3}{12EI}\left(\frac{3}{4}L^2 - a_3^2\right)$$
$$= \frac{2.10\text{ kips}\left(12½\text{ ft}\right)\left(12\text{ in./ft}\right)^3}{12\left(29{,}000\text{ ksi}\right)\left(612\text{ in.}^4\right)}\left[\frac{3}{4}\left(30\text{ ft}\right)^2 - \left(12½\text{ ft}\right)^2\right]$$
$$= 0.111\text{ in.}$$

For both roll-bracing beams 10 ft from the beam end (a_{RB} = 10 ft),

$$\Delta_{pRB} = \frac{P_{RB}a_{RB}}{12EI}\left(\frac{3}{4}L^2 - a_{RB}^2\right)$$
$$= \frac{0.410\text{ kip}\left(10\text{ ft}\right)\left(12\text{ in./ft}\right)^3}{12\left(29{,}000\text{ ksi}\right)\left(612\text{ in.}^4\right)}\left[\frac{3}{4}\left(30\text{ ft}\right)^2 - \left(10\text{ ft}\right)^2\right]$$
$$= 0.0191\text{ in.}$$

The total flexural deformation of spandrel beam is,

$$\Delta_f = \Delta_w + 2\left(\Delta_{p1} + \Delta_{p2} + \Delta_{p3} + \Delta_{RB}\right)$$
$$= 0.296\text{ in.} + 2\left(0.0285\text{ in.} + 0.0791\text{ in.} + 0.111\text{ in.} + 0.0191\text{ in.}\right)$$
$$= 0.770\text{ in.}$$

The rotation of the spandrel beam at midspan (between roll beams) is,

$$\theta = \frac{2F\theta ta^2}{GJ} = \frac{2\left(0.037\text{ rad.}\right)\left(0.350\text{ kip-ft/ft}\right)\left(67.8\text{ in.}\right)^2}{\left(11{,}200\text{ ksi}\right)\left(0.810\text{ in.}^4\right)}$$
$$= 0.0131\text{ rad. (or } 0.751°\text{)}$$

The rotation of the roll beam (see Figure 6-18) is,

$$\theta_{RB} = \frac{2TL_{RB}}{3EI_{xRB}} = \frac{2\left(1.75\text{ kip-ft}\right)\left(10\text{ ft}\right)\left(12\text{ in./ft}\right)^2}{3\left(29{,}000\text{ ksi}\right)\left(53.8\text{ in.}^4\right)}$$
$$= 0.00108\text{ rad. (or } 0.617°\text{)}$$

The rotation of spandrel beam at midspan including the rotation contribution from the end of the roll beams is,

$$\theta_{total} = \theta + \theta_{RB}$$
$$= 0.751° + 0.0617°$$
$$= 0.813°$$

Assuming no load sharing between the spandrel and the adjacent beam, the total vertical deflection of the adjacent roof beam at the roll beam location due to the roll beam reaction is,

$$\Delta_{adj} = \frac{P_{RB}s_{RB}}{24EI_{xa}}\left(3L^2 - 4s_{RB}^2\right)$$
$$= \frac{0.410\text{ kip}\left(10\text{ ft}\right)\left(12\text{ in./ft}\right)^3}{24\left(29{,}000\text{ ksi}\right)\left(301\text{ in.}^4\right)}\left[3\left(30\text{ ft}\right)^2 - 4\left(10\text{ ft}\right)^2\right]$$
$$= 0.0778\text{ in.}$$

The rotational deformation of the spandrel is calculated, as illustrated in Figure 6-19. The initial horizontal and vertical distances between the centroid of the W18×40 and the bottom of the curtain wall support angle are,

x_i = 6 in.
y_i = 0 in.

The distance between the beam centroid and the curtain wall attachment point is,

$$r = \sqrt{x_i^2 + y_i^2}$$
$$= \sqrt{\left(6\text{ in.}\right)^2 + \left(0\text{ in.}\right)^2}$$
$$= 6.00\text{ in.}$$

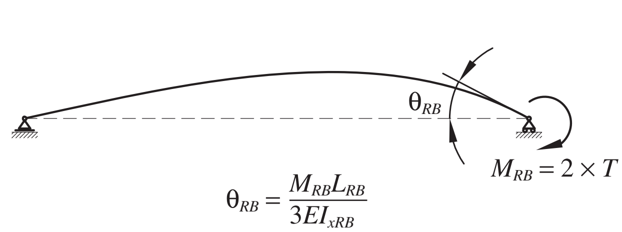


Fig. 6-18. Rotation of roll beam.

The initial angle of the curtain wall centroid with respect to the horizontal is,

$$\begin{aligned}\alpha &= \tan^{-1}\left(\frac{y_i}{x_i}\right)\\ &= \tan^{-1}\left(\frac{0\text{ in.}}{6\text{ in.}}\right)\\ &= 0°\end{aligned}$$

The final angle of the curtain wall attachment after rotation is,

$$\begin{aligned}\beta &= \alpha - \theta_{total}\\ &= 0° - 0.813°\\ &= -0.813°\end{aligned}$$

The final horizontal and vertical curtain wall position with respect to the beam centroid is,

$$\begin{aligned}x_f &= r\cos(\beta)\\ &= (6.00\text{ in.})\cos(-0.813°)\\ &= 6.00\text{ in.}\end{aligned}$$

$$\begin{aligned}y_f &= r\sin(\beta)\\ &= (6.00\text{ in.})\sin(-0.813°)\\ &= -0.0851\text{ in.}\end{aligned}$$

The out-of-plane movement of the angle due to beam rotation is,

$$\begin{aligned}\Delta_{hrb} &= x_f - x_i\\ &= 6.00\text{ in.} - 6.00\text{ in.}\\ &= 0\text{ in. (small enough that it is negligible)}\end{aligned}$$

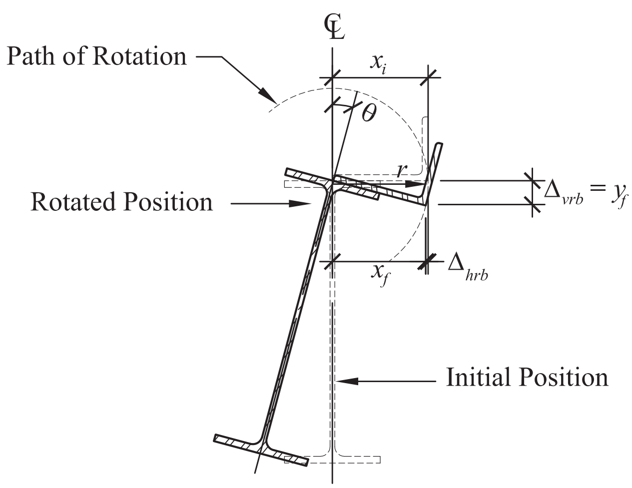


Fig. 6-19. Rigid body rotation about center of top flange.

The rigid-body translation at the centerline of the curtain wall due to rotation of the spandrel is,

$$\begin{aligned}\Delta_{vrb} &= y_i - y_f\\ &= 0\text{ in.} - (-0.0851\text{ in.})\\ &= 0.0851\text{ in.}\end{aligned}$$

The total vertical movement at the curtain wall attachment point (the sum of the flexural and rotational deflections) is,

$$\begin{aligned}\Delta_v &= \Delta_f + \Delta_{vrb}\\ &= 0.770 + 0.0851\\ &= 0.855\text{ in.} \le 0.90\text{ in.} \quad \textbf{o.k.}\end{aligned}$$

For the design of the roll beam connection to the spandrel, the vertical shear force on the bolt group is equal to the reaction at the end of the roll beam,

$$V_{uRB} = R_{Tu} = 0.406\text{ kip}$$

The required flexural strength of the bolt group is,

$$\begin{aligned}M_{uRB} &= 2T_u\\ &= 2(2.03\text{ kip-ft})\\ &= 4.06\text{ kip-ft}\end{aligned}$$

The eccentricity on the bolt group is,

$$\begin{aligned}e_{bolt} &= \frac{M_{uRB}}{V_{uRB}}\\ &= \frac{4.06\text{ kip-ft}(12\text{ in./ft})}{0.406\text{ kips}}\\ &= 120\text{ in.}\end{aligned}$$

The eccentricity on the bolts is larger than the data tabulated in AISC *Manual* Table 7-7. In this case, the elastic method can be used as illustrated in Figure 6-20. Alternatively, the instantaneous center method and an iterative calculation similar to that used to determine the values in AISC *Manual* Table 7-7 can be used. The polar moment of inertia of the bolt group is,

$$\begin{aligned}J_{bolt} &= 2s_{max}^2\\ &= 2(3\text{ in.})^2\\ &= 18.0\text{ in.}^2\end{aligned}$$

The maximum horizontal shear force due to torsion on a single bolt is,

$$r_{ut} = \frac{M_{uRB} s_{max}}{J_{bolt}}$$
$$= \frac{4.06 \text{ kip-ft}\left(3 \text{ in.}\right)\left(12 \text{ in./ft}\right)}{18.0 \text{ in.}^2}$$
$$= 8.12 \text{ kips}$$

The maximum vertical shear force on a single bolt is,

$$r_{ut} = \frac{V_{uRB}}{n}$$
$$= \frac{0.406 \text{ kip}}{3}$$
$$= 0.135 \text{ kip}$$

The resultant maximum shear force for a single bolt is,

$$r_u = \sqrt{r_{ut}^2 + r_{uv}}$$
$$= \sqrt{\left(8.12 \text{ kips}\right)^2 + \left(0.135 \text{ kips}\right)^2}$$
$$= 8.12 \text{ kips}$$

From AISC Manual Table 7-1, the available strength of a ¾-in.-diameter ASTM A325-N bolt in single shear is,

$\phi_v r_n = 15.9 \text{ kips} > r_u$ **o.k.**

Other checks for this connection, including the strength of the plate for bolt bearing and the strength of the weld to the beam, are required, but left as an exercise for the reader.

Comments:
The use of roll beams substantially reduces the torsional rotation of the spandrel beam, which allows the use of lighter, shallower spandrels, particularly for spandrels with longer spans.

The effects of torsional continuity in the spandrel beam at the roll beams are ignored in this example. When desired, these effects can be considered, and may reduce the torsional rotations of the spandrel beam further.

If the top flange of the W10×12 roll beam is flush with the top of the spandrel beam and its web is coped to facilitate the connection to the spandrel beam, it may be difficult to install three bolts in the full-depth stiffener. The designer could consider dropping the top flange of the roll beam or using a deeper section (such as a W12).

Example 6.3—Roof Spandrel Beam with Eccentric Curtain Wall Load—Torsion Avoided with HSS and Roll Beams

Repeat Example 6.2, but with an HSS5×3×¼ spanning as shown in Figures 6-21 and 6-22 between W12×16 roll beams.

Given:
An ASTM A500 Grade B HSS5×3×¼ resists all torsion associated with the curtain wall support, and its span for torsion, L_T = 10 ft. The architect has set the edge angle face at 1 ft 3 in. from the centerline of the spandrel beam; thus, l_{eod} = 15 in.

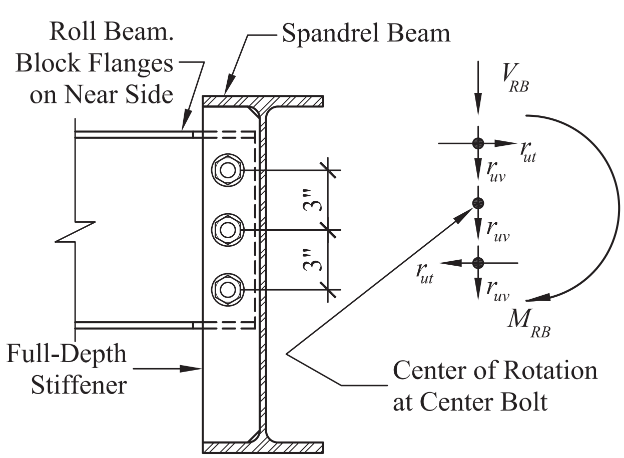


Fig. 6-20. Forces on bolts at roll beam to spandrel beam connection.

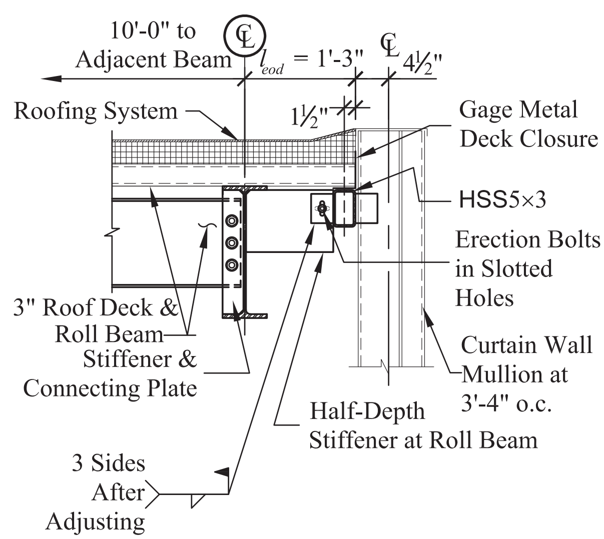


Fig. 6-21. Section of roof spandrel beam with edge HSS and roll beams.

The loading is similar to that given previously, but there are eight façade attachments at spacing s_c = 3 ft 4 in. = 3.33 ft, centered along the length of the spandrel beam. The eccentricity between the center-of-gravity of the curtain wall and the centerline of the spandrel beam, e_{cw} = 19.5 in. The eccentricity between the curtain wall and the centerline of the HSS, e'_{cw} = 6 in.

Wind loads are transmitted through the HSS directly to the roof deck, and thus impose negligible torsional loading on the structure. (Also, the HSS need not be checked for weak-axis bending. However, the designer should give close attention to the roof deck design.)

The roll beam is bolted to a ⅜-in.-thick ASTM A36 full-depth stiffener on the spandrel (see Figure 6-20). There are three ⅞-in.-diameter ASTM A325-N bolts with spacing, s = 3 in. The distance from the centroid of the bolt group to the farthest bolt, s_{max} = 3 in.

The W18×50 spandrel has the following properties:

d = 18.0 in.
t_w = 0.355 in.
t_f = 0.570 in.
b_f = 7.50 in.
I_x = 800 in.^{4}
S_x = 88.9 in.^{3}
Z_x = 101 in.^{3}

The W12×16 roll beams have the following property:

I_x = 103 in.^{4}

The HSS5×3×¼ has the following properties:

w_{HSS} = 12.18 lb/ft
t_{des} = 0.233 in.
I_x = 10.7 in.^{4}
I_y = 4.81 in.^{4}
Z_x = 5.38 in.^{3}
Z_y = 3.77 in.^{3}
J = 11.0 in.^{4}
C = 6.10 in.^{3}

Solution:
The loading is as illustrated in Figure 6-23. The roof width tributary to the spandrel beam for dead and snow loads is,

$$\begin{aligned} t_r &= \frac{s_{deck}}{2} + l_{eod} \\ &= \frac{10 \text{ ft}}{2} + \frac{15 \text{ in.}}{12 \text{ in./ft}} \\ &= 6.25 \text{ ft} \end{aligned}$$

The uniformly distributed dead load on the spandrel beam is,

$$\begin{aligned} w_D &= w_{bm} + w_{dead} t_r \\ &= 0.050 \text{ kip/ft} + 0.015 \text{ kip/ft}^2 \left(6.25 \text{ ft}\right) \\ &= 0.144 \text{ kip/ft} \end{aligned}$$

The uniformly distributed snow load on the spandrel beam is,

$$\begin{aligned} w_S &= w_{snow} t_r \\ &= 0.030 \text{ kip/ft}^2 \left(6.25 \text{ft}\right) \\ &= 0.188 \text{ kip/ft} \end{aligned}$$

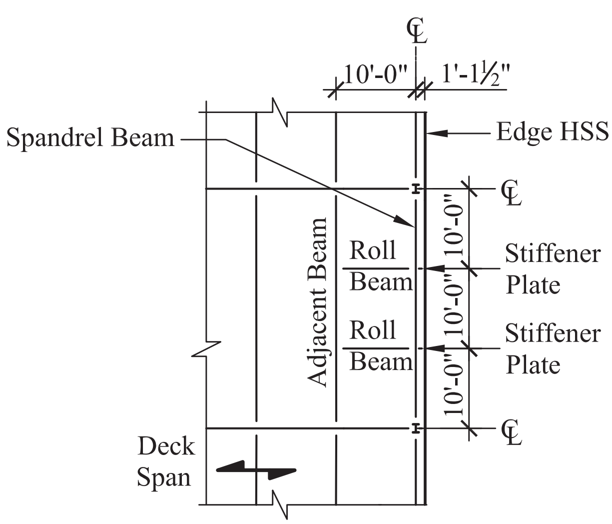


Fig. 6-22. Roof plan at spandrel beam.

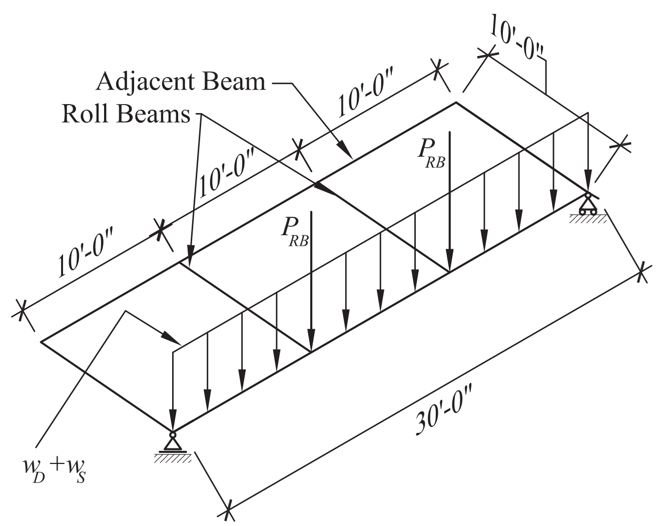


Fig. 6-23. Loads on spandrel beam.

The factored uniform load is,

$$
\begin{aligned}
w_u &= 1.2w_D + 1.6w_S \\
&= 1.2(0.144 \text{ kip/ft}) + 1.6(0.188 \text{ kip/ft}) \\
&= 0.474 \text{ kip/ft}
\end{aligned}
$$

The half-depth plate between the spandrel and the HSS has the following depth,

$$
\begin{aligned}
d_{pl} &= \frac{d}{2} - t_f \\
&= \frac{18.0 \text{ in.}}{2} - 0.570 \text{ in.} \\
&= 8.43 \text{ in.}
\end{aligned}
$$

An 8½-in. plate depth will be used.

The HSS is supported at each column and each roll beam, and must resist the vertical load and torsion associated with the curtain wall attachment loads, as shown in Figure 6-24. For simplicity of calculation, it will be assumed that the HSS is torsionally restrained at each roll beam.

Each wind point load on the HSS is,

$$
\begin{aligned}
P_{wind} &= p_w\left(\frac{h}{2}\right)s_c \\
&= 0.030 \text{ kip/ft}^2\left(\frac{14 \text{ ft}}{2}\right)(3.33 \text{ ft}) \\
&= 0.700 \text{ kip}
\end{aligned}
$$

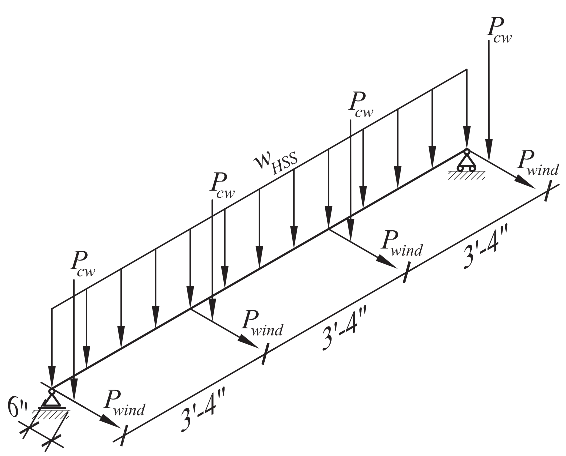


Fig. 6-24. Loads on edge HSS between roll beams.

Each curtain wall gravity point load on the HSS is,

$$
\begin{aligned}
P_{cw} &= w_{curt}h_c s_c \\
&= 0.015 \text{ kip/ft}^2(28 \text{ ft})(3.33 \text{ ft}) \\
&= 1.40 \text{ kips}
\end{aligned}
$$

The torsional moment at each curtain wall point load is,

$$
\begin{aligned}
T &= P_{cw}e'_{cw} \\
&= 1.40 \text{ kips}\left(\frac{6 \text{ in.}}{12 \text{ in./ft}}\right) \\
&= 0.700 \text{ kip-ft}
\end{aligned}
$$

The factored loads and moment are,

$$
\begin{aligned}
P_{u\,w} &= 0.8P_{wind} \\
&= 0.8(0.700 \text{ kip}) \\
&= 0.560 \text{ kip}
\end{aligned}
$$

$$
\begin{aligned}
P_{u\,cw} &= 1.2P_{cw} \\
&= 1.2(1.40 \text{ kips}) \\
&= 1.68 \text{ kips}
\end{aligned}
$$

$$
\begin{aligned}
T_u &= 1.2T \\
&= 1.2(0.700 \text{ kip-ft}) \\
&= 0.840 \text{ kip-ft}
\end{aligned}
$$

The uniformly distributed load on the HSS due to self weight is,

$$
\begin{aligned}
w_{u\,HSS} &= 1.2w_{HSS} \\
&= 1.2(0.01218 \text{ kip/ft}) \\
&= 0.0146 \text{ kip/ft}
\end{aligned}
$$

Using AISC Manual Table 3-22c to calculate the moment at the interior support of a continuous beam with three equal spans, the strong-axis required flexural strength is,

$$
\begin{aligned}
M_{ux} &= 0.267P_{u\,cw}L_T + 0.10w_{u\,HSS}L_T^2 \\
&= 0.267(1.68 \text{ kips})(10 \text{ ft}) \\
&\quad + 0.10(0.0146 \text{ kip/ft})(10 \text{ ft})^2 \\
&= 4.63 \text{ kip-ft}
\end{aligned}
$$

The required shear strength for the HSS is,

$$
\begin{aligned}
V_{ux} &= 1.27P_{u\,cw} + 0.6w_{u\,HSS}L_T \\
&= 1.27(1.68 \text{ kips}) + 0.6(0.0146 \text{ kip/ft})(10 \text{ ft}) \\
&= 2.22 \text{ kips}
\end{aligned}
$$

From AISC Manual Table 1-11, the HSS wall slenderness ratio $h/t_w = 18.5$. Because this is less than $2.45\sqrt{E/F_y}$, per AISC Specification Section H3.1, the torsional design strength is,

$$\begin{aligned}\phi_T T_n &= \phi_T F_{cr} C \\ &= \frac{0.90(0.6)(46\text{ ksi})(6.10\text{ in.}^3)}{12\text{ in./ft}} \\ &= 12.6\text{ kip-ft}\end{aligned}$$

The shear area for strong-axis shear is,

$$\begin{aligned}A_{wx} &= 2ht_{des} \\ &= 2(d-3t_{des})(t_{des}) \\ &= 2[5\text{ in.}-3(0.233\text{ in.})](0.233\text{ in.}) \\ &= 2.00\text{ in.}^2\end{aligned}$$

For unstiffened webs with $h/t_w < 260,\ k_v = 5$

$$\begin{aligned}1.10\sqrt{k_v\left(\frac{E}{F_y}\right)} &= 1.10\sqrt{5\left(\frac{29{,}000\text{ ksi}}{46\text{ ksi}}\right)} \\ &= 61.8\end{aligned}$$

Because $h/t_w < 61.8$, $C_v = 1.0$. From AISC Specification Section G2.1(b) the strong-axis design shear strength is,

$$\begin{aligned}\phi_v V_{nx} &= \phi_v 0.6F_y A_{wx} C_v \\ &= 1.00(0.6)(46\text{ ksi})(2.00\text{ in.}^2)(1.0) \\ &= 55.2\text{ kips}\end{aligned}$$

Flange local buckling and web local buckling do not control the flexural strength of the HSS used. Therefore, the design flexural strength of the HSS is based on the limit state of yielding,

$$\begin{aligned}\phi_b M_{nx} &= \phi_b F_y Z_x \\ &= \frac{0.90(46\text{ ksi})(5.38\text{ in.}^3)}{12\text{ in./ft}} \\ &= 18.5\text{ kip-ft}\end{aligned}$$

The ratio of required torsional strength to design torsional strength per the AISC Specification Section H3.2 is,

$$\begin{aligned}\frac{T_u}{\phi_T T_n} &= \frac{0.840\text{ kip-ft}}{12.6\text{ kip-ft}} \\ &= 0.0667\end{aligned}$$

Because this ratio is less than 20%, torsional effects can be neglected and the available flexural strength is,

$$\phi_b M_{nx} = 18.5\text{ kip-ft} > M_{ux} \qquad \textbf{o.k.}$$

The available shear strength is,

$$\phi_v V_{nx} = 5.22\text{ kips} > V_{ux} \qquad \textbf{o.k.}$$

As with the previous example, the roll beams resist the twist of the cladding system and, in doing so, impose downward vertical loads on the spandrel beam. The loads on the spandrel beam from the HSS can be obtained using the coefficients from the continuous beam tables presented in AISC Manual Table 3-22c. The point loads on the spandrel at the one-third points (at the roll beams) are calculated as illustrated in Figure 6-25. The service vertical reaction on the spandrel beam due to the self-weight of the HSS is,

$$\begin{aligned}R_{HSS} &= \frac{11}{10}(w_{HSS}L_T) \\ &= \frac{11}{10}(0.01218\text{ kip/ft})(10\text{ ft}) \\ &= 0.134\text{ kip}\end{aligned}$$

The service vertical reaction on the spandrel beam due to the curtain wall is,

$$\begin{aligned}R_{cw} &= 2.27P_{cw} + P_{cw} \\ &= 2.27(1.40\text{ kips}) + 1.40\text{ kips} \\ &= 4.58\text{ kips}\end{aligned}$$

Note, with two intermediate curtain wall point loads and one more at each roll beam, the torsional moment resisted by each roll beam is $3T$.

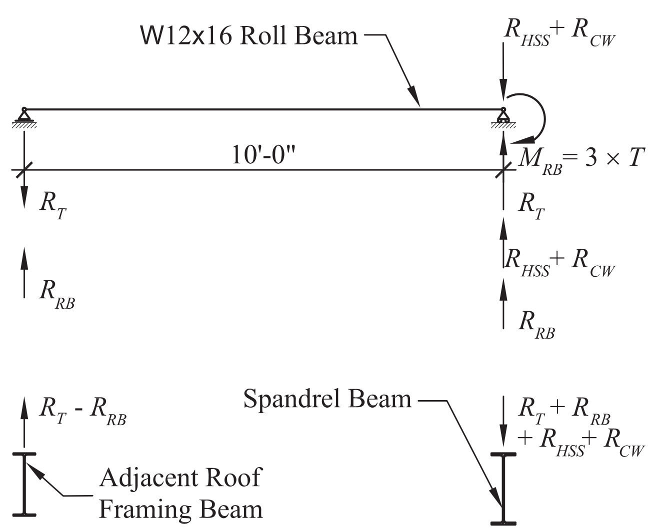


Fig. 6-25. Loads on and reactions due to roll beam.

The service moment on each roll beam is,

$$M_{RB} = \left(R_{HSS} + R_{cw}\right)\left(l_{eod} - \frac{B}{2}\right) + 3T$$

$$= \frac{\left(0.134 \text{ kip} + 4.58 \text{ kips}\right)\left(15 \text{ in.} - \dfrac{3 \text{ in.}}{2}\right)}{12 \text{ in./ft}}$$

$$+ 3\left(0.700 \text{ kip-ft}\right)$$

$$= 7.40 \text{ kip-ft}$$

The service vertical reaction on the spandrel beam due to the end moment on the roll beam is,

$$R_T = \frac{M_{RB}}{L_{RB}}$$

$$= \frac{7.40 \text{ kip-ft}}{10 \text{ ft}}$$

$$= 0.740 \text{ kip}$$

The service vertical reaction on the spandrel beam due to the self-weight of the roll beam is,

$$R_{RB} = w_{RB}\frac{L_{RB}}{2}$$

$$= 0.016 \text{ kip/ft}\left(\frac{10 \text{ ft}}{2}\right)$$

$$= 0.0800 \text{ kip}$$

The service point load on the spandrel beam at each roll beam is,

$$P_{RB} = R_{HSS} + R_{cw} + R_T + R_{RB}$$

$$= 0.134 \text{ kip} + 4.58 \text{ kips} + 0.740 \text{ kip} + 0.0800 \text{ kip}$$

$$= 5.53 \text{ kips}$$

The factored point load on the spandrel beam at each roll beam is,

$$P_{RBu} = 1.2P_{RB}$$

$$= 1.2\left(5.53 \text{ kips}\right)$$

$$= 6.64 \text{ kips}$$

The required shear strength of the spandrel beam at each end is,

$$V_u = \frac{w_u L}{2} + P_{RBu}$$

$$= \frac{0.474 \text{ kip/ft}\left(30 \text{ ft}\right)}{2} + 6.64 \text{ kips}$$

$$= 13.7 \text{ kips}$$

The required flexural strength of the spandrel beam at midspan is,

$$M_u = \frac{w_u L^2}{8} + P_{RBu}L_T$$

$$= \frac{0.474 \text{ kip/ft}\left(30 \text{ ft}\right)^2}{8} + \left(6.64 \text{ kips}\right)\left(10 \text{ ft}\right)$$

$$= 120 \text{ kip-ft}$$

The available flexural strength is,

$$\phi_b M_n = \phi_b F_y Z_x$$

$$= \frac{0.90\left(50 \text{ ksi}\right)\left(101 \text{ in.}^3\right)}{12 \text{ in./ft}}$$

$$= 379 \text{ kip-ft} > M_u \quad \textbf{o.k.}$$

From AISC *Specification* Sections G1 and G2.1 (and using ϕ_v = 1.0 and C_v = 1.0 because this W-shape has $h/t_w \le 2.24\sqrt{E/F_y}$), the available shear strength is,

$$\phi_v V_n = \phi_v 0.6F_y d t_w C_v$$

$$= 1.00\left(0.6\right)\left(50 \text{ ksi}\right)\left(18.0 \text{ in.}\right)\left(0.355 \text{ in.}\right)\left(1.0\right)$$

$$= 192 \text{ kips} > V_u \quad \textbf{o.k.}$$

The deflection of the HSS and spandrel beam is as shown in Figure 6-26. The vertical deflection at the midspan of the HSS is calculated assuming that it is simply supported at each roll beam. This assumption is conservative, and could be improved upon with more refined analysis. Deflection at the HSS midspan with point loads at the one-third points of the span (using AISC *Manual* Table 3-23, cases 9 and 1) is,

$$\Delta_{HSS} = \frac{P_{cw}L_T^3}{28EI_x} + \frac{5w_{HSS}L_T^4}{384EI_x}$$

$$= \frac{\left(1.40 \text{ kips}\right)\left(10 \text{ ft}\right)^3\left(12 \text{ in./ft}\right)^3}{28\left(29{,}000 \text{ ksi}\right)\left(10.7 \text{ in.}^4\right)}$$

$$+ \frac{5\left(0.01218 \text{ kip/ft}\right)\left(10 \text{ ft}\right)^4\left(12 \text{ in./ft}\right)^3}{384\left(29{,}000 \text{ ksi}\right)\left(10.7 \text{ in.}^4\right)}$$

$$= 0.287 \text{ in.}$$

The torsional rotation of the HSS at midspan is,

$$\theta_{HSS} = \frac{T\left(\dfrac{L_T}{3}\right)}{GJ}$$

$$= \frac{0.700 \text{ kip-ft}\left(\dfrac{10 \text{ ft}}{3}\right)\left(12 \text{ in./ft}\right)^2}{11{,}200 \text{ ksi}\left(11.0 \text{ in.}^4\right)}$$

$$= 0.00273 \text{ rad.} \left(\text{or } 0.156^\circ\right)$$

The vertical displacement of the HSS due to rotation of the HSS between roll beams at midspan is,

$$\Delta_{HSS\theta} = \theta_{HSS}\left(\frac{B}{2}\right)$$
$$= 0.00273\text{ rad.}\left(\frac{3\text{ in.}}{2}\right)$$
$$= 0.00409\text{ in.}$$

This deformation is small because the HSS is a closed shape and is stiff in torsion. Thus, this deflection is neglected in the rest of this example. The vertical deflection of the spandrel beam at the roll beam (reduced from AISC *Manual* Table 3-23, cases 9 and 1) is,

$$\Delta_s = \frac{5P_{RB}L^3}{162EI_x} + \frac{22\left(w_D + w_S\right)L^4}{1{,}944EI_x}$$
$$= \frac{5\left(5.53\text{ kips}\right)\left(30\text{ ft}\right)^3\left(12\text{ in./ft}\right)^3}{162\left(29{,}000\text{ ksi}\right)\left(800\text{ in.}^4\right)}$$
$$+ \frac{22\left(0.144\text{ kip/ft} + 0.188\text{ kip/ft}\right)\left(30\text{ ft}\right)^4\left(12\text{ in./ft}\right)^3}{1{,}944\left(29{,}000\text{ ksi}\right)\left(800\text{ in.}^4\right)}$$
$$= 0.570\text{ in.}$$

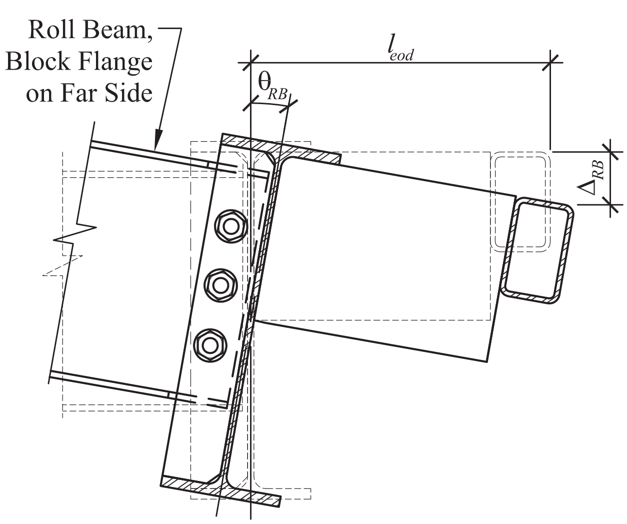


Fig. 6-26. Rotation of spandrel at roll beam.

The rotation of the roll beam due to the eccentricity of the edge HSS (see Figure 6-27) is,

$$\theta_{RB} = \frac{M_{RB}L_{RB}}{3EI_{xRB}}$$
$$= \frac{7.40\text{ kip-ft}\left(10\text{ ft}\right)\left(12\text{ in./ft}\right)^2}{3\left(29{,}000\text{ ksi}\right)\left(103\text{ in.}^4\right)}$$
$$= 0.00119\text{ rad. (or }0.0681°\text{)}$$

The rigid body translation of the corner of the HSS due to rotation of the roll beam assuming a small angle of rotation (where $\theta = \sin\theta$) is,

$$\Delta_{RB} = \theta_{RB}l_{eod}$$
$$= 0.00119\text{ rad.}\left(15\text{ in.}\right)$$
$$= 0.0179\text{ in.}$$

This deformation is typically small. It has been neglected in the rest of this example. The total vertical displacement of the HSS at center of the span is limited to 0.90 in. in Example 6.1,

$$\Delta_{total} = \Delta_{HSS} + \Delta_s$$
$$= 0.287\text{ in.} + 0.570\text{ in.}$$
$$= 0.857\text{ in.} \le 0.90 \quad \textbf{o.k.}$$

The design of the roll beam connection to the spandrel beam (see Figure 6-28) begins with the calculation of the factored moment on each roll beam,

$$M_{uRB} = 1.2M_{RB}$$
$$= 1.2\left(7.40\text{ kip-ft}\right)$$
$$= 8.88\text{ kip-ft}$$

The factored vertical shear on the bolt group is,

$$V_{uRB} = 1.2R_T$$
$$= 1.2\left(0.740\text{ kip}\right)$$
$$= 0.888\text{ kip}$$

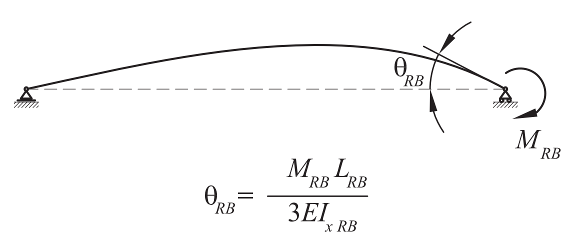


Fig. 6-27. Rotation of roll beam.

The eccentricity on the bolt group is,

$$\begin{aligned} e_{bolt} &= \frac{M_{uRB}}{V_{uRB}} \\ &= \frac{8.88 \text{ kip-ft}(12 \text{ in./ft})}{0.888 \text{ kip}} \\ &= 120 \text{ in.} \end{aligned}$$

The eccentricity on the bolts is sufficiently large that it exceeds the parameters that are tabulated in AISC Manual Table 7-7 for eccentrically loaded bolt groups. In this case, the elastic method is applied. Alternatively, the instantaneous center method and an iterative calculation similar to that used to determine the values in AISC Manual Table 7-7 can be used.

The polar moment of inertia of the bolt group is,

$$\begin{aligned} J_{bolt} &= 2s_{\max}^2 \\ &= 2(3 \text{ in.})^2 \\ &= 18.0 \text{ in.}^2 \end{aligned}$$

The maximum horizontal shear force due to torsion on a single bolt is,

$$\begin{aligned} r_{ut} &= \frac{M_{uRB}\, s_{\max}}{J_{bolt}} \\ &= \frac{8.88 \text{ kip-ft}(3 \text{ in.})(12 \text{ in./ft})}{18.0 \text{ in.}^2} \\ &= 17.8 \text{ kips} \end{aligned}$$

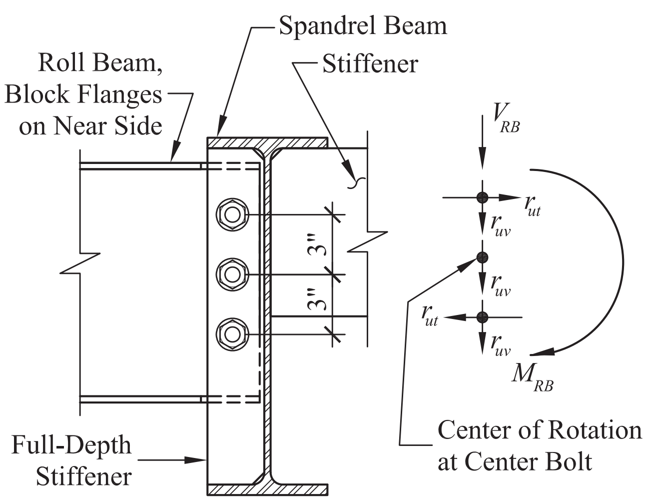


Fig. 6-28. Forces on bolts at roll beam to spandrel beam connection.

The maximum vertical shear on a single bolt is,

$$\begin{aligned} r_{uv} &= \frac{V_{uRB}}{n} \\ &= \frac{0.888 \text{ kip}}{3} \\ &= 0.296 \text{ kip} \end{aligned}$$

The resultant required shear strength on a single bolt is,

$$\begin{aligned} r_u &= \sqrt{r_{ut}^2 + r_{uv}^2} \\ &= \sqrt{(17.8 \text{ kips})^2 + (0.296 \text{ kip})^2} \\ &= 17.8 \text{ kips} \end{aligned}$$

The strength of one ⅞-in.-diameter A325-N bolt in single shear (AISC Manual, Table 7-1) is,

$$\phi_v r_n = 21.6 \text{ kips} > r_u \qquad \textbf{o.k.}$$

For the design of the stiffener plate, the unbraced length for bending is,

$$\begin{aligned} L_{bPL} &= l_{eod} - B \\ &= 15 \text{ in.} - 3 \text{ in.} \\ &= 12 \text{ in.} \end{aligned}$$

The slenderness of the plate for yielding and lateral-torsional buckling limit states per AISC Specification Section F11 is,

$$\frac{L_{bPL} d_{PL}}{t_{PL}^2} = \frac{12 \text{ in.}(8\tfrac{1}{2} \text{ in.})^2}{(\tfrac{3}{8} \text{ in.})^2} = 725$$

The limiting plate slenderness for yielding is,

$$\frac{0.08E}{F_y} = \frac{0.08(29{,}000 \text{ ksi})}{36 \text{ ksi}} = 64.4$$

The limiting plate slenderness for lateral-torsional buckling is,

$$\frac{1.9E}{F_y} = \frac{1.9(29{,}000 \text{ ksi})}{36 \text{ ksi}} = 1{,}530$$

The values of M_y and M_p are,

$$\begin{aligned} M_y &= F_y S_x \\ &= F_y\left(\frac{t_{PL} d_{PL}^2}{6}\right) \\ &= 36 \text{ ksi}\left(\frac{\tfrac{3}{8} \text{ in.}(8\tfrac{1}{2} \text{ in.})^2}{6}\right)\left(\frac{1 \text{ ft}}{12 \text{ in.}}\right) \\ &= 13.6 \text{ kip-ft} \end{aligned}$$

$$M_p = F_y\left(\frac{t_{PL}d_{PL}^2}{4}\right)$$

$$= 36\left(\frac{\frac{3}{8}\text{ in.}\left(8\frac{1}{2}\text{ in.}\right)^2}{4}\right)\left(\frac{1\text{ ft}}{12\text{ in.}}\right)$$

$$= 20.3\text{ kip-ft}$$

The plate available flexural strength (assuming $C_b = 1.0$) is,

$$\phi_b M_n = 0.90\left[1.52 - 0.274\left(\frac{L_{bPL}d_{PL}}{t_{PL}^2}\right)\frac{F_{yPL}}{E}\right]M_y \le \phi_b M_p$$

$$= 0.90\left[1.52 - 0.274\left(\frac{12\text{ in.}\left(8\frac{1}{2}\text{ in.}\right)}{\left(\frac{3}{8}\text{ in.}\right)^2}\right)\right.$$

$$\left.\times\left(\frac{36\text{ ksi}}{29{,}000\text{ ksi}}\right)\right]\left(13.6\text{ kip-ft}\right)$$

$$\le 0.90(20.3\text{ kip-ft})$$

$$= 15.6\text{ kip-ft} \le 18.3\text{ kip-ft}$$

$$= 15.6\text{ kip-ft} > M_{uRB}\quad \textbf{o.k.}$$

The ⅜-in.-thick plate is adequate for flexure.

Comments:
A similar plate would also be required at the columns at each end of the 30-ft span. In a complete design, the designer would also check other limit states, such as the attachment of this plate to the columns, the shear strength of the plate, the strength of the roll beam, the strength of the adjacent roof beam, the bolt bearing strength of the plate, and sizing the weld between the plate and the beam. The designer should also consider the effects of the unbraced bottom flange of the roll beam on its flexural strength.

Example 6.4—Roof Spandrel Beam with Eccentric Curtain Wall—Torsion on Spandrel Avoided by Kickers

Reconsider the 30-ft-long spandrel beam investigated in Example 6.3 with the curtain wall supported at s_c = 3 ft 4 in. on center. In this case, however, use single-angle diagonal kickers spaced at 10 ft on center along the spandrel to brace the stiffener plates. The curtain wall is supported by an HSS spanning between the stiffener plates as illustrated in Figures 6-29 and 6-30. The same load and deflection criteria apply as in the previous example. The roof deck is typically welded to the spandrel beam with ⅝-in.-diameter puddle welds at 12 in. on center.

Check the strength of the edge HSS, spandrel beam, and kicker considering torsional effects. Also, check the deflection of the spandrel beam and edge HSS assembly at the beam midspan considering torsional effects. The welds between the roof deck and the spandrel beam and adjacent beam to resist the horizontal component of the load in the kicker and the bending of the stiffener plate must be considered.

Given:
For this example assume the controlling load combination for strength is $1.2D + 1.6S + 0.8W$ ($D + S + W$ for deflection). The HSS resists all torsion associated with the curtain wall support and its span, L_T = 10 ft. The eccentricity between the curtain wall and the centerline of the HSS, e'_{cw} = 6 in.

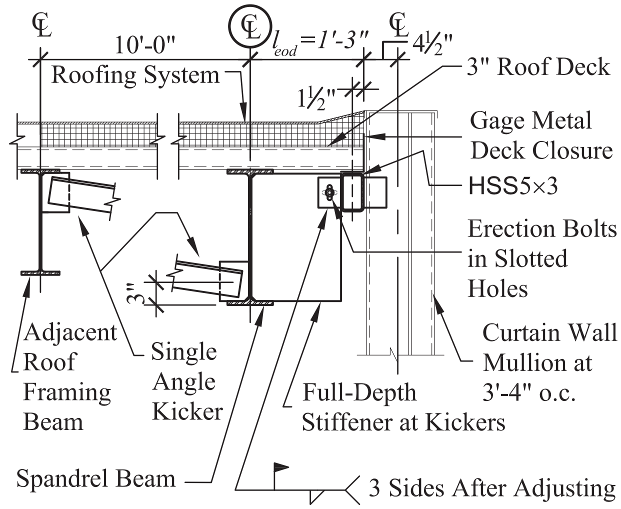


Fig. 6-29. Sections of roof spandrel beam with edge HSS *and kickers.*

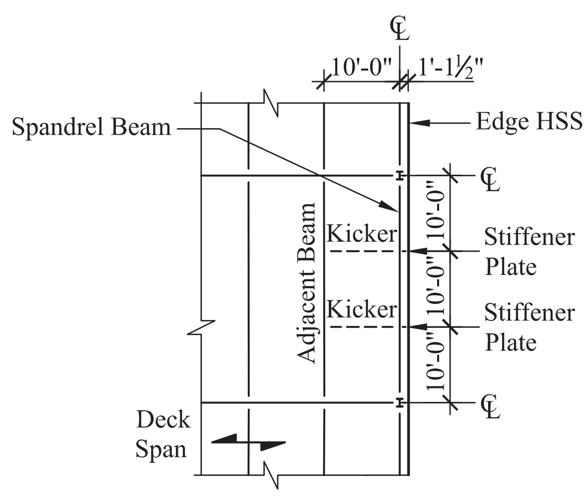


Fig. 6-30. Roof plan at spandrel beam.

Assume the spandrel beam is pinned for flexure at the supporting columns and the flexural length, $L = 30$ ft. The story height, $h = 14$ ft. The height of cladding suspended from the spandrel beam, $h_c = 28$ ft. The horizontal length of the kickers, $L_K = 10$ ft and the vertical distance between the bottom flange of the spandrel beam and the centroid of the kickers, $c = 3$ in.

The roof deck braces the compression flange of the spandrel beam against lateral-torsional buckling. The roof deck span, $s_{deck} = 10$ ft. The distance from the center of the spandrel beam to the edge of the roof deck, $l_{eod} = 1\text{ft } 3\text{in.} = 15$ in. This causes a horizontal eccentricity between the curtain wall and the centerline of the beam, $e_{cw} = l_{eod} + 4.5 \text{ in.} = 19.5$ in.

Wind imposes negligible torsion loads on the structure. Wind loads are transmitted through the stiffener plate and spandrel top flange into the roof deck.

The roof dead load, $w_{dead} = 15$ psf. The uniform roof snow load, $w_{snow} = 30$ psf. The uniform curtain wall load, $w_{curt} = 15$ psf. The wind pressure, $p_w = 30$ psf. Refer to Figure 6-31.

The W18×50 spandrel beam has the following properties:

$$
\begin{aligned}
d &= 18.0 \text{ in.} \\
t_w &= 0.355 \text{ in.} \\
b_f &= 7.50 \text{ in.} \\
t_f &= 0.570 \text{ in.} \\
I_x &= 800 \text{ in.}^4 \\
Z_x &= 101 \text{ in.}^3 \\
S_x &= 88.9 \text{ in.}^3
\end{aligned}
$$

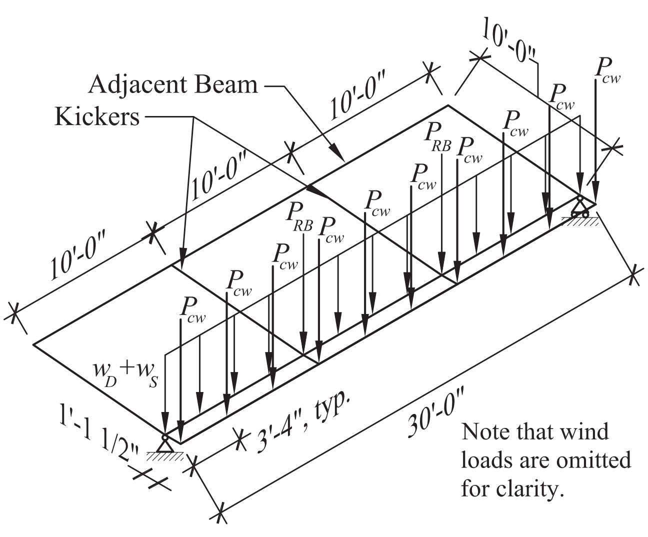


Fig. 6-31. Loads on spandrel beam and edge HSS.

The L3½×3½×5⁄16 kickers have the following properties:

$$
\begin{aligned}
A &= 2.09 \text{ in.}^2 \\
w_K &= 7.20 \text{ lb/ft}
\end{aligned}
$$

The 3 in., 20 Ga. ASTM A653 roof deck has the following properties:

$$
\begin{aligned}
t_{deck} &= 0.0358 \text{ in.} \\
F_u &= 45 \text{ ksi}
\end{aligned}
$$

The edge HSS5×3×¼ ASTM A500 Grade B has the following properties:

$$
\begin{aligned}
I_x &= 10.7 \text{ in.}^4 \\
J &= 11.0 \text{ in.}^4 \\
w_{HSS} &= 12.18 \text{ lb/ft}
\end{aligned}
$$

Solution:

The roof tributary width for the spandrel beam is,

$$
\begin{aligned}
t_r &= \frac{s_{deck}}{2} + l_{eod} \\
&= \frac{10 \text{ ft}}{2} + \frac{15 \text{ in.}}{12 \text{ in./ft}} \\
&= 6.25 \text{ ft}
\end{aligned}
$$

The uniform dead load on the beam is,

$$
\begin{aligned}
w_D &= w_{bm} + w_{dead} t_r \\
&= 0.050 \text{ kip/ft} + 0.015 \text{ kip/ft}^2 \left(6.25 \text{ ft}\right) \\
&= 0.144 \text{ kip/ft}
\end{aligned}
$$

The uniform snow load on the beam is,

$$
\begin{aligned}
w_S &= w_{snow} t_r \\
&= 0.030 \text{ kip/ft}^2 \left(6.25 \text{ ft}\right) \\
&= 0.188 \text{ kip/ft}
\end{aligned}
$$

The depth of the stiffener plate is,

$$
\begin{aligned}
d_{PL} &= d - 2t_f \\
&= 18.0 \text{ in.} - 2\left(0.570 \text{ in.}\right) \\
&= 16.9 \text{ in.}
\end{aligned}
$$

See Example 6.3 for determination of the strength of the HSS and the strength of the spandrel beam. The compression strength of the kicker is checked as follows (see Figure 6-32). The service curtain wall gravity point load on the HSS is,

$$
\begin{aligned}
P_{cw} &= w_{curt} h_c s_c \\
&= 0.015 \text{ kip/ft}^2 \left(28 \text{ ft}\right)\left(\frac{40 \text{ in.}}{12 \text{ in./ft}}\right) \\
&= 1.40 \text{ kips}
\end{aligned}
$$

The service torsion at each end of the HSS span (L_T) is,

$$T = P_{cw}e'_{cw}$$
$$= 1.40 \text{ kips}\left(\frac{6 \text{ in.}}{12 \text{ in./ft}}\right)$$
$$= 0.700 \text{ kip-ft}$$

The service vertical reaction on the spandrel beam due to the self-weight of the HSS is,

$$R_{HSS} = (11/10)\, w_{HSS}L_T$$
$$= (11/10)(0.01218 \text{ kip/ft})(10 \text{ ft})$$
$$= 0.134 \text{ kip}$$

The service vertical reaction on the spandrel beam due to the curtain wall is,

$$R_{cw} = 2.27P_{cw} + P_{cw}$$
$$= 2.27(1.40 \text{ kips}) + 1.40 \text{ kips}$$
$$= 4.58 \text{ kips}$$

The service torsional moment at each kicker is,

$$M_K = (R_{HSS} + R_{cw})\left(l_{eod} - \frac{B}{2}\right) + 3T$$
$$= (0.134 \text{ kip} + 4.58 \text{ kips})\frac{\left(15 \text{ in.} - \dfrac{3 \text{ in.}}{2}\right)}{12 \text{ in./ft}} + 3(0.700 \text{ kip-ft})$$
$$= 7.40 \text{ kip-ft}$$

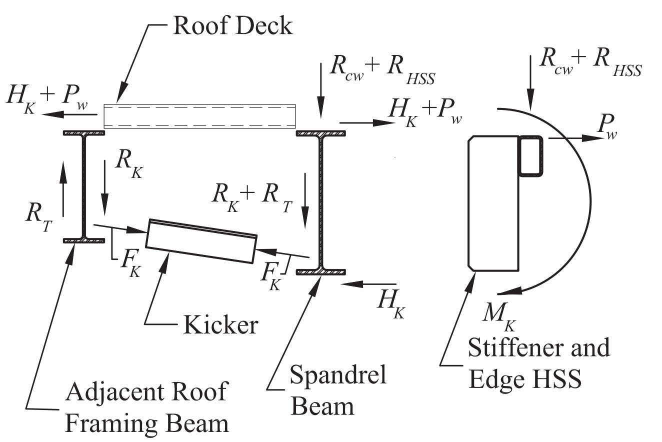


Fig. 6-32. Calculation of force in kicker.

The service vertical reaction on the spandrel due to torsional moment at the kicker is,

$$R_T = \frac{M_K}{L_K}$$
$$= \frac{7.40 \text{ kip-ft}}{10 \text{ ft}}$$
$$= 0.740 \text{ kip}$$

The length of the kicker is,

$$L'_K = \sqrt{L_K^2 + (d-c)^2}$$
$$= \sqrt{(10 \text{ ft})^2 + \left(\frac{17.9 \text{ in.} - 3 \text{ in.}}{12 \text{ in./ft}}\right)^2}$$
$$= 10.1 \text{ ft}$$

The service vertical reaction on the spandrel due to self-weight of the kicker is,

$$R_K = w_K \frac{L'_K}{2}$$
$$= 0.00720 \text{ kip/ft}\left(\frac{10.1 \text{ ft}}{2}\right)$$
$$= 0.0364 \text{ kip}$$

This reaction is small and has been neglected in the remainder of this example.

The service point load on the spandrel at each kicker is,

$$P_K = R_{HSS} + R_{cw} + R_T$$
$$= 0.134 \text{ kip} + 4.58 \text{ kips} + 0.740 \text{ kip}$$
$$= 5.45 \text{ kips}$$

The service horizontal component of the force in the kicker is,

$$H_K = \frac{M_K}{d-c}$$
$$= \frac{7.40 \text{ kip-ft}(12 \text{ in./ft})}{17.9 \text{ in.} - 3 \text{ in.}}$$
$$= 5.96 \text{ kips}$$

The service axial force in the kicker is,

$$F_K = \frac{L'_K}{L_K}H_K$$
$$= \frac{10.1}{10}(5.96 \text{ kips})$$
$$= 6.02 \text{ kips}$$

The required kicker axial strength is,

$$\begin{aligned} F_{uK} &= 1.2F_K \\ &= 1.2(6.02\text{ kips}) \\ &= 7.22\text{ kips} \end{aligned}$$

From AISC Manual Table 4-12, the available axial compression strength of the single angle, assume that the kicker has an effective length factor of $K = 1.0$ with the ends pinned is,

$$\phi_c P_n = 9.82\text{ kips} > F_{uK} \quad \textbf{o.k.}$$

The deflection of the assembly at the center of the span is calculated as the vertical deflection at the HSS midspan assuming that it is simply supported at each kicker. The deflection at the center of the HSS span with point loads at one-third points of the span is derived from AISC Manual Table 3-23, case 9.

$$\begin{aligned} \Delta_{HSS} &= \frac{P_{cw}L_T^3}{28EI_x} + \frac{5w_{HSS}L_T^4}{384EI_x} \\ &= \frac{(1.40\text{ kips})(10\text{ ft})^3(12\text{ in./ft})^3}{28(29{,}000\text{ ksi})(10.7\text{ in.}^4)} \\ &\quad + \frac{5(0.01218\text{ kip/ft})(10\text{ ft})^4(12\text{ in./ft})^3}{384(29{,}000\text{ ksi})(10.7\text{ in.}^4)} \\ &= 0.287\text{ in.} \end{aligned}$$

Torsional rotation of the HSS at midspan is,

$$\begin{aligned} \theta_{HSS} &= \frac{T\left(\dfrac{L_T}{3}\right)}{GJ} \\ &= \frac{0.700\text{ kip-ft}\left(\dfrac{10\text{ ft}}{3}\right)(12\text{ in./ft})^2}{11{,}200\text{ ksi}(11.0\text{ in.}^4)} \\ &= 0.00273\text{ rad. (or } 0.156°) \end{aligned}$$

The vertical displacement of the HSS due to rotation of the HSS between kickers at midspan is,

$$\begin{aligned} \Delta_{HSS\theta} &= \theta_{HSS}\left(\frac{B}{2}\right) \\ &= 0.00273\text{ rad.}\left(\frac{3\text{ in.}}{2}\right) \\ &= 0.00409\text{ in.} \end{aligned}$$

This deflection is small and has been neglected in the rest of this example.

The vertical deflection of the spandrel beam at the kicker (reduced from the AISC Manual Table 3-23, cases 1 and 9) is,

$$\begin{aligned} \Delta_s &= \frac{5P_KL^3}{162EI_x} + \frac{22(w_D + w_S)L^4}{1{,}944EI_x} \\ &= \frac{5(5.45\text{ kips})(30\text{ ft})^3(12\text{ in./ft})^3}{162(29{,}000\text{ ksi})(800\text{ in.}^4)} \\ &\quad + \frac{22(0.144\text{ kip/ft} + 0.188\text{ kip/ft})(30\text{ ft})^4(12\text{ in./ft})^3}{1{,}944(29{,}000\text{ ksi})(800\text{ in.}^4)} \\ &= 0.565\text{ in.} \end{aligned}$$

The axial shortening of the kicker due to the compression load is,

$$\begin{aligned} \Delta'_K &= \frac{F_K L'_K}{A_K E} \\ &= \frac{6.02\text{ kips}(10.1\text{ ft})(12\text{ in./ft})}{2.09\text{ in.}^2(29{,}000\text{ ksi})} \\ &= 0.0120\text{ in.} \end{aligned}$$

This deflection is small and has been neglected in the rest of this example.

The service wind point load transmitted to the roof deck at the stiffener is calculated using the reaction at an interior support of a three span beam (AISC Manual, Table 3-22c),

$$\begin{aligned} \mathrm{P}_{wind} &= \frac{11}{10}p_w\left(\frac{\mathrm{h}}{2}\right)L_T \\ &= 1.1(0.030\text{ kip/ft}^2)\left(\frac{14\text{ ft}}{2}\right)(10\text{ ft}) \\ &= 2.31\text{ kips} \end{aligned}$$

Based on how much deck is tributary to each kicker, the effective width of the metal roof deck is,

$$b_{eff} = 3\text{ ft}$$

The horizontal elongation of the metal roof deck is,

$$\begin{aligned} \Delta_D &= \frac{(H_K + P_{wind})s_{deck}}{b_{eff}t_{deck}E} \\ &= \frac{(5.96\text{ kips} + 2.31\text{ kips})(10\text{ ft})}{3\text{ ft}(0.0358\text{ in.})(29{,}000\text{ ksi})} \\ &= 0.0266\text{ in.} \end{aligned}$$

Using Δ_D as the lateral translation of the top flange of the spandrel beam, the rotation of the spandrel beam at the kicker due to load in the roof deck is,

$$\theta_K = \frac{\Delta_D}{d}$$
$$= \frac{0.0266 \text{ in.}}{18.0 \text{ in.}}$$
$$= 0.00149 \text{ rad. (or } 0.0850°)$$

The rigid body translation of the corner of the HSS due to rotation of the kicker assuming small angle rotation (where $\theta = \sin\theta$) is,

$$\Delta_K = \theta_K l_{eod}$$
$$= 0.00149 \text{ rad.} \left(15 \text{ in.}\right)$$
$$= 0.0224 \text{ in.}$$

Both Δ_D and Δ_K are small and will be neglected.

From Example 6.1, the vertical deflection is limited to 0.90 in. The total vertical displacement of the HSS at the center of its span is,

$$\Delta_{total} = \Delta_{HSS} + \Delta_s$$
$$= 0.287 \text{ in.} + 0.565 \text{ in.}$$
$$= 0.852 \text{ in.} \le 0.90 \text{ in.} \quad \textbf{o.k.}$$

For the welds from the roof deck to the spandrel beam and adjacent beams, try seven deck-to-beam puddle welds within the deck effective width ($n_{weld} = 7$). Note that this is more welding than the typical deck perimeter attachment. The weld size is $d_{weld} = \frac{5}{8}$ in.

The factored shear per weld is,

$$V_{uw} = \frac{1.2H_K + 0.8P_{wind}}{n_{weld}}$$
$$= \frac{1.2\left(5.96 \text{ kips}\right) + 0.8(2.31 \text{ kips})}{7}$$
$$= 1.29 \text{ kips}$$

The weld shear strength in accordance with the Steel Deck Institute Diaphragm Design Manual (SDI, 2004), page 4-4 is,

$$\phi = 0.70$$
$$Q_f = 2.2\, t_{deck} F_u \left(d_{weld} - t_{deck}\right)$$
$$= 2.2\left(0.0358 \text{ in.}\right)\left(45 \text{ ksi}\right)\left(0.625 \text{ in.} - 0.0358 \text{ in.}\right)$$
$$= 2.09 \text{ kips}$$
$$\phi Q_f = 0.70\left(2.09 \text{ kips}\right)$$
$$= 1.46 \text{ kips} > V_{uw}$$

Seven puddle welds between the roof deck and the spandrel beam within a 3-ft width adjacent to the stiffeners and kickers will carry the forces from torsion and wind loads from the curtain wall into the roof deck. The same number of welds is also required at the adjacent roof framing beam (the first interior beam). The design of the stiffener plate was illustrated in Example 6.3.

Comments:

The edge HSS allows for a substantial eccentricity between the cladding and the spandrel framing without a substantial penalty on the weight of the framing. In this example, using kickers in place of slightly heavier roll beams has reduced the steel tonnage. However, the kickers require additional puddle welds between the roof deck and the spandrel and adjacent roof framing beams than might otherwise be required. The connections between the kicker and the adjacent beams are most often field-welded and these welds can add some cost to the project. The best choice between these various options depends upon finding the right mix of material costs and labor costs. Thus, it is ideal to discuss the options with the steel fabricator to determine which one will provide the least total cost.

Example 6.5—Floor Spandrel Beam with Eccentric Precast Panel Loads

The 30-ft-long spandrel beam shown in Figures 6-33 and 6-34 supports the floor framing shown in addition to 6-in.-thick precast concrete cladding panels weighing 75 psf. The panels are supported by steel brackets bearing on each floor slab 2 in. from the edge of the slab. The 6¼-in.-thick floor system consists of 3¼ in. of 3,000 psi lightweight concrete on 3-in. metal floor deck. The floor live load is 100 psf, and the superimposed (post-composite) dead load is 15 psf. The maximum vertical deflection due to superimposed loads is ½ in.

Design a bearing connection for the precast panel by evaluating the slab for flexural loads induced by the precast cladding. If the slab cannot resist the flexural loads, evaluate whether the bent-plate pour stop is capable of resisting the flexural loads.

Determine the total vertical deflection at the support points for the precast panels. Also, determine the adequacy of the spandrel beam including the effects of normal stresses due to torsional warping.

Given:

For this example assume the controlling load combination for strength is $1.2D + 1.6S + 0.8W$ ($D + S + W$ for deflection). The floor framing beams perpendicular to the spandrel provide some torsional restraint for the spandrel. The contribution of the composite floor system to the torsional stiffness of the spandrel is conservatively ignored. The horizontal

eccentricity between the centroid of the precast panel and the edge of the pour stop is resolved by a couple between adjacent floors.

The spandrel beam is a W24×76 with a length of $L = 30$ ft for flexure and a length of $L_T = 10$ ft for torsion. The span of the perpendicular floor framing beams is $L_{fb} = 25$ ft with the spacing of the supports for the precast panels at $s_p = 10$ ft.

The post-composite properties of the spandrel beam are, $S_{eff} = 217$ in.^{3} and $I_{eff} = 3{,}410$ in.^{4} The pre-composite factored moment demand $M_{pre} = 93.0$ kip-ft (with a load factor of 1.2) with a total factored moment demand $M_u = 414$ kip-ft. The post-composite deflection of the spandrel beam due to live load and superimposed dead load at mid-span is 0.377 in.

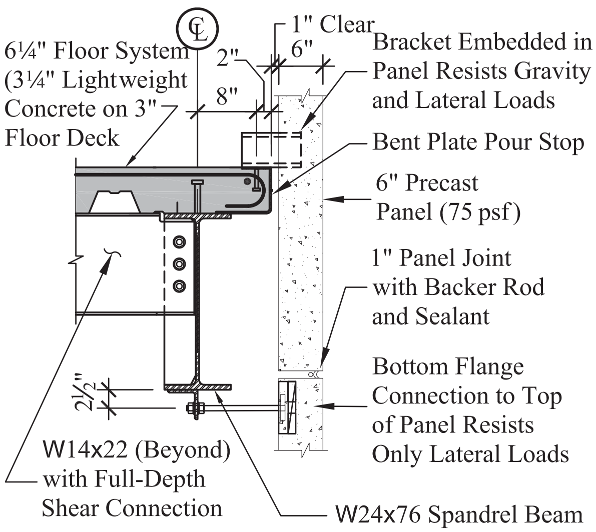


Fig. 6-33. Section of floor spandrel beam with precast panel supported on slab.

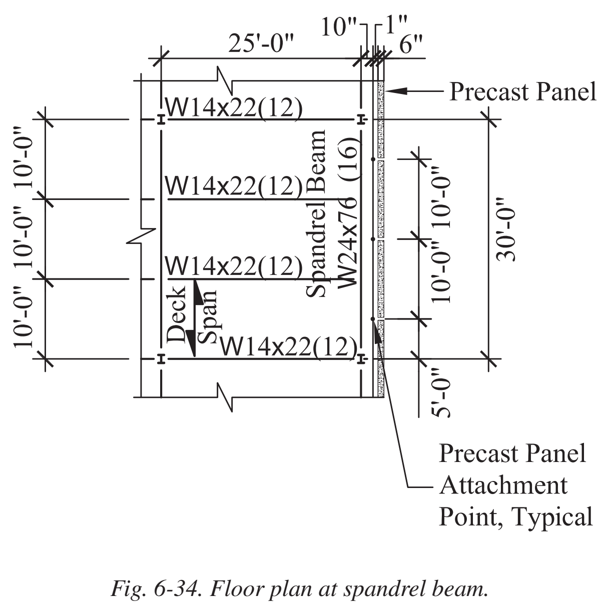


Fig. 6-34. Floor plan at spandrel beam.

The story height is $h = 12$ ft, with the eccentricity between the point of precast load and the centerline of the beam, $e = 8$ in. The horizontal eccentricity between the point of the precast load and the centerline of the precast panels, $e_p = 6$ in. The vertical eccentricity between the bottom flange of the spandrel beam and the point of horizontal panel load, $e_r = 2\frac{1}{2}$ in.

The width of the precast embed parallel to the slab edge, $b_{pc} = 3$ in.

The W24×76 spandrel beam has the following properties:

$$d = 23.9 \text{ in.}$$
$$b_f = 8.99 \text{ in.}$$
$$t_f = 0.680 \text{ in.}$$

$$I_y = 82.5 \text{ in.}^4$$
$$S_x = 176 \text{ in.}^3$$
$$S_y = 18.4 \text{ in.}^3$$

$$J = 2.68 \text{ in.}^4$$
$$a = 104 \text{ in.}$$
$$W_{no} = 52.2 \text{ in.}^2$$

The W14×22 floor framing beams have $I_x = 199$ in.^{4}

The pour stop plate is made from ASTM A36 material.

Solution:

For the loads on the spandrel beam, see Figures 6-35, 6-36, and 6-37. The service precast panel point load at each support point on the spandrel is,

$$\begin{aligned} P_{pc} &= \left(0.075 \text{ kip/ft}^2\right) h s_p \\ &= \left(0.075 \text{ kip/ft}^2\right)\left(12 \text{ ft}\right)\left(10 \text{ ft}\right) \\ &= 9.00 \text{ kips} \end{aligned}$$

The factored precast panel point load at each support point on the spandrel beam is,

$$\begin{aligned} P_{upc} &= 1.2 P_{pc} \\ &= 1.2\left(9.00 \text{ kips}\right) \\ &= 10.8 \text{ kips} \end{aligned}$$

The service wind point load on the spandrel beam at each precast panel support point location is,

$$\begin{aligned} P_w &= p_w s_p h \\ &= 0.030 \text{ kip/ft}^2 \left(10 \text{ ft}\right)\left(12 \text{ ft}\right) \\ &= 3.60 \text{ kips} \end{aligned}$$

The factored wind point load at each precast panel support point on the spandrel is,

$$
\begin{aligned}
P_{uw} &= 0.8P_w \\
&= 0.8(3.60 \text{ kips}) \\
&= 2.88 \text{ kips}
\end{aligned}
$$

The service horizontal force on the spandrel beam due to eccentricity of the panel with respect to the pour stop is,

$$
\begin{aligned}
P_H &= \frac{P_{pc}e_p}{h-d-e_r} \\
&= \frac{9.00 \text{ kips}(6 \text{ in.})}{12 \text{ ft}(12 \text{ in./ft}) - 23.9 \text{ in.} - 2.5 \text{ in.}} \\
&= 0.459 \text{ kip}
\end{aligned}
$$

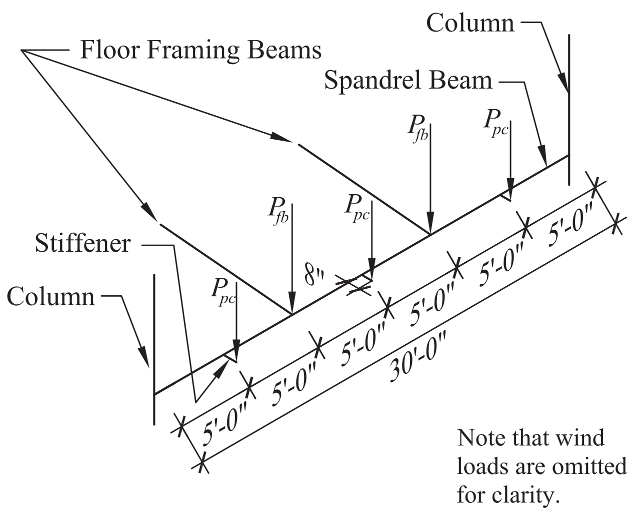


Fig. 6-35. Loads on spandrel beam.

The factored horizontal force on the spandrel beam due to eccentricity of the panel with respect to the pour stop is,

$$
\begin{aligned}
P_{uH} &= 1.2(P_H) \\
&= 1.2(0.459 \text{ kip}) \\
&= 0.551 \text{ kip}
\end{aligned}
$$

For the design of the bearing support for the precast panel, the slab strength to support the gravity load of the precast panel will be evaluated assuming that the panel load spreads over an effective width at 45° to each side.

The effective width of the slab to resist the precast load is,

$$
\begin{aligned}
b_{eff} &= 2\tan 45°\left(e - \frac{b_f}{2}\right) + b_{pc} \\
&= 2\tan 45°\left(8 \text{ in.} - \frac{8.99 \text{ in.}}{2}\right) + 3 \text{ in.} \\
&= 10.0 \text{ in.}
\end{aligned}
$$

The factored moment in the slab due to the precast gravity load is,

$$
\begin{aligned}
M_{us} &= P_{upc}\left(e - \frac{b_f}{2}\right)\frac{b}{b_{eff}} \\
&= 10.8 \text{ kips}\left(8 \text{ in.} - \frac{8.99 \text{ in.}}{2}\right)\frac{12 \text{ in./ft}}{10.0 \text{ in.}} \\
&= 45.4 \text{ kip-in./ft}
\end{aligned}
$$

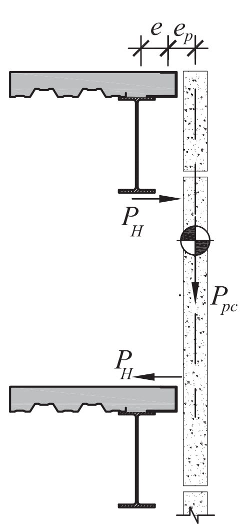


Fig. 6-36. Resolution of precast panel eccentricity.

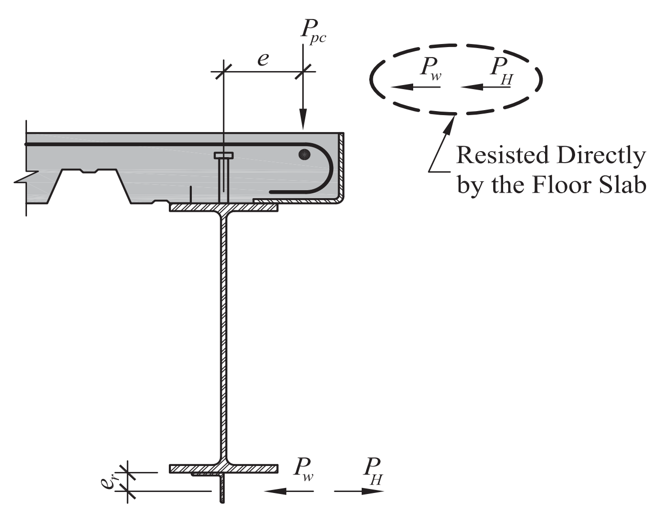


Fig. 6-37. Vertical and lateral forces on spandrel beam.

From Table 5-4, it will be difficult to provide enough reinforcement in the slab to resist the flexure created by the panel load. As an alternative, determine if the bent plate can resist all flexure induced by the precast panels (see Figure 6-38). The horizontal force is resisted by the studs in tension and developed into the slab through the slab reinforcing steel. For flexure, AISC Specification Section F11 provides the flexural strength of rectangular bars bent about the minor axis.

Try a ½-in.-thick bent plate where the effective width of the plate tributary to the precast attachment, assuming the load spreads at 45° to each side, is,

$$\begin{aligned} b_{eff} &= 2\tan 45^\circ\left(e - \frac{b_f}{2}\right) + b_{pc} \\ &= 2\tan 45^\circ\left(8\text{ in.} - \frac{8.99\text{ in.}}{2}\right) + 3\text{ in.} \\ &= 10.0\text{ in.} \end{aligned}$$

The moment on the plate is,

$$M_{up} = M_{us} = 45.4\text{ kip-in./ft}$$

The plastic section modulus of the plate per foot is,

$$\begin{aligned} Z_p &= \frac{bt^2}{4} \\ &= \frac{12\text{ in./ft }(0.5\text{ in.})^2}{4} \\ &= 0.750\text{ in.}^3\text{/ft} \end{aligned}$$

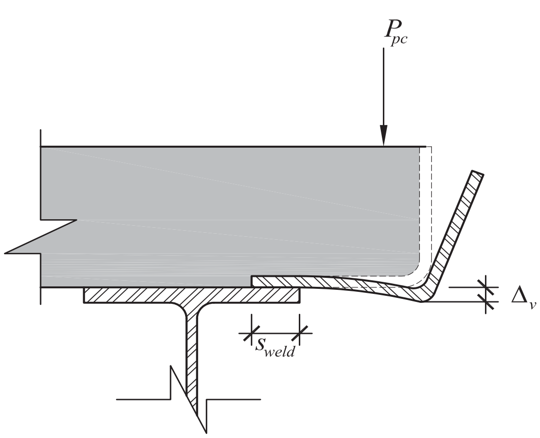


Fig. 6-38. Deflection of bent plate pour stop.

The available flexural strength of the bent plate is,

$$\begin{aligned} \phi M_{np} &= \phi F_y Z_p \\ &= 0.90(36\text{ ksi})\left(0.750\text{ in.}^3\text{/ft}\right) \\ &= 24.3\text{ kip-in./ft} < M_{us} \quad \textbf{n.g.} \end{aligned}$$

The precast panel loads are too large for a reasonable thickness bent plate to cantilever off of the spandrel beam flange. The ½-in. plate alone does not have adequate flexural strength. Additionally, as shown below, the deflection of the plate (which combines with the deflection of the spandrel) is excessive.

The distance between welds connecting the plate to the beam flange is,

$$s_{weld} = 2\text{ in.}$$

The effective moment of inertia of the plate is,

$$\begin{aligned} I &= \frac{b_{eff}t^3}{12} \\ &= \frac{10.0\text{ in.}(0.50\text{ in.})^3}{12} \\ &= 0.104\text{ in.}^4 \end{aligned}$$

The vertical deflection of the end of the plate, neglecting the weight of the concrete (from AISC Manual Table 3-23, case 26), is,

$$\begin{aligned} \Delta_V &= \frac{P_{pc}e^2}{3EI}\left(e + s_{weld}\right) \\ &= \frac{9.00\text{ kips }(8\text{ in.})^2}{3(29{,}000\text{ ksi})(0.104\text{ in}^4)}(8\text{ in.} + 2\text{ in.}) \\ &= 0.637\text{ in.} \end{aligned}$$

Therefore, stiffener plates must be added to the spandrel at each precast panel support point and the spandrel designed to take torsion. See Figure 6-39. Adding the stiffeners eliminates the need to design the bent plate pour stop for any loads other than the wet load of the concrete. From Table 5-11, a ½-in.-thick bent plate pour stop will be sufficient.

For the purposes of the analysis, consider the spandrel beam restrained for torsion at each of the perpendicular floor framing beams and the columns. Use the provisions from AISC Design Guide No. 9 to determine the maximum angle of twist. Assume the rotation is about the center of the top flange of the spandrel beam since it is restrained by the concrete slab.

The service torsional moment at the midspan of L_T due to the precast load is,

$$
\begin{aligned}
T_{pc} &= P_{pc}e - P_H\left(d + e_r\right) \\
&= \frac{9.00\text{ kips}\left(8\text{ in.}\right) - 0.459\text{ kip}\left(23.9\text{ in.} + 2\tfrac{1}{2}\text{ in.}\right)}{12\text{ in./ft}} \\
&= 4.99\text{ kip-ft}
\end{aligned}
$$

The service torsional moment at the midspan of L_T due to the wind load applied at the bottom of the beam is,

$$
\begin{aligned}
T_w &= \frac{P_w}{2}\left(d\right) \\
&= \frac{\left(\dfrac{3.60\text{ kips}}{2}\right)\left(23.9\text{ in.}\right)}{12\text{ in./ft}} \\
&= 3.59\text{ kip-ft}
\end{aligned}
$$

The total service torsional moment applied at the midspan of L_T is,

$$
\begin{aligned}
T &= T_{pc} + T_w \\
&= 4.99\text{ kip-ft} + 3.59\text{ kip-ft} \\
&= 8.58\text{ kip-ft}
\end{aligned}
$$

The service torsion at each end of L_T is,

$$
\begin{aligned}
T_e &= \frac{T}{2} \\
&= \frac{8.58\text{ kip-ft}}{2} \\
&= 4.29\text{ kip-ft}
\end{aligned}
$$

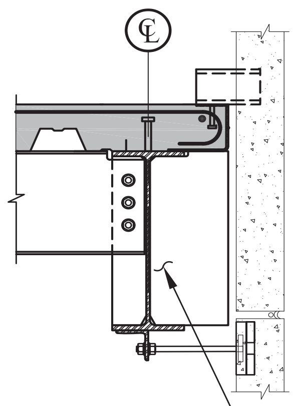


Fig. 6-39. Section of spandrel beam with stiffener plate at bottom flange connection to precast panel.

To obtain the service moment applied to the ends of the floor-framing beams, multiply T_e by 2 because the precast panels are symmetric about the floor framing beams. The service moment applied to the end of the floor framing beam is,

$$
\begin{aligned}
M_{fb} &= 2T_e \\
&= 2\left(4.29\text{ kip-ft}\right) \\
&= 8.58\text{ kip-ft}
\end{aligned}
$$

The rotation at the end of the floor framing beam (see Figure 6-40) is,

$$
\begin{aligned}
\theta_{fb} &= \frac{M_{fb}L_{fb}}{3EI_{x\,fb}} \\
&= \frac{8.58\text{ kip-ft}\left(25\text{ ft}\right)\left(12\text{ in./ft}\right)^2}{3\left(29{,}000\text{ ksi}\right)\left(199\text{ in.}^4\right)} \\
&= 0.00178\text{ rad.}\quad(\text{or } 0.102^\circ)
\end{aligned}
$$

The service vertical reaction on the spandrel beam due to the end moment on the floor framing beam is,

$$
\begin{aligned}
R_{fb} &= \frac{M_{fb}}{L_{fb}} \\
&= \frac{8.58\text{ kip-ft}}{25\text{ ft}} \\
&= 0.343\text{ kip}
\end{aligned}
$$

In this case, R_{fb} is small. In many practical cases it may be neglected.

$$
\frac{L_T}{a} = \frac{10\text{ ft}}{104\text{ in./}\left(12\text{in./ft}\right)} = 1.15
$$

From AISC Design Guide No. 9, Appendix B, case 3 for $\alpha = 0.5$, the torsional function at the center of the span L_T (between roll beams) is,

$$
\theta\left(\frac{GJ}{TL_T}\right) = 0.025\text{ rad.}
$$

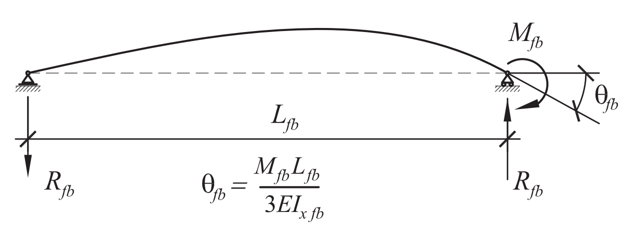


Fig. 6-40. Rotation of roll beam.

The rotation at the center of the span, L_T, is,

$$\begin{aligned}\theta &= \frac{(0.025\,\text{rad.})TL_T}{GJ}\\ &= \frac{(0.025\,\text{rad.})(8.58\text{ kip-ft})(10\text{ ft})(12\text{ in./ft})^2}{(11{,}200\text{ ksi})(2.68\text{ in.}^4)}\\ &= 0.0102\text{ rad. (or } 0.585°)\end{aligned}$$

Note that this torsional rotation θ assumes the spandrel beam can rotate about its centroid. In reality, the spandrel beam is restrained by the slab and will rotate about the center of its top flange. Thus, the actual rotation will be slightly smaller.

Adding the rotation of the spandrel beam between the roll beams to the rotation of the roll beams,

$$\begin{aligned}\theta_{total} &= \theta + \theta_{fb}\\ &= 0.585° + 0.102°\\ &= 0.687°\end{aligned}$$

The vertical deflection at the precast support point due to rigid body rotation,

$$\begin{aligned}\Delta_{vrb} &= e\sin(\theta_{total})\\ &= (8\text{ in.})\sin(0.687°)\\ &= 0.0959\text{ in.}\end{aligned}$$

The total vertical deflection is,

$$\begin{aligned}\Delta_{total} &= \Delta_{vrb} + \Delta_{pcl}\\ &= 0.0959\text{ in.} + 0.377\text{ in.}\\ &= 0.473\text{ in.} \le 1/2\text{ in.} \quad \textbf{o.k.}\end{aligned}$$

The available strength of the spandrel beam is calculated using case 3 from AISC Design Guide No. 9, Appendix B to determine the normal stresses due to warping. The factored torsional moment at the middle of the torsion span is,

$$\begin{aligned}T_u &= 1.2T_{pc} + 0.8T_w\\ &= 1.2(4.99\text{ kip-ft}) + 0.8(3.59\text{ kip-ft})\\ &= 8.86\text{ kip-ft}\end{aligned}$$

The torsional function at the center of the span L_T (between roll beams) is,

$$\theta_u''\left(\frac{GJa}{T_u}\right) = 0.25$$

$$\begin{aligned}\theta_u'' &= \frac{0.25T_u}{GJa}\\ &= \frac{0.25(8.86\text{ kip-ft})(12\text{ in./ft})}{(11{,}200\text{ ksi})(2.68\text{ in.}^4)(104\text{ in.})}\\ &= 8.51\times10^{-6}\text{ rad./in.}^2\end{aligned}$$

The normal stress at midspan due to warping is,

$$\begin{aligned}\sigma_{wsu} &= EW_{no}\theta_u''\\ &= 29{,}000\text{ ksi}(52.2\text{ in.}^2)(8.51\times10^{-6}\text{ rad./in.}^2)\\ &= 12.9\text{ ksi}\end{aligned}$$

The factored post composite moment demand is,

$$\begin{aligned}M_{post} &= M_u - M_{pre}\\ &= 414\text{ kip-ft} - 93.0\text{ kip-ft}\\ &= 321\text{ kip-ft}\end{aligned}$$

The interaction of combined normal stresses at the midspan of the beam is based on the AISC Design Guide No. 9, Equation 4.16a, using the elastic combination of stresses,

$$\begin{aligned}\sigma_{total} &= \frac{M_{pre}}{S_x} + \frac{M_{post}}{S_{eff}} + \sigma_{wsu}\\ &= \frac{93\text{ kip-ft}(12\text{ in./ft})}{176\text{ in.}^3}\\ &\quad + \frac{321\text{ kip-ft}(12\text{ in./ft})}{217\text{ in.}^3} + 12.9\text{ ksi}\\ &= 37.0\text{ ksi}\end{aligned}$$

$$\begin{aligned}\phi F_y &= 0.90F_y\\ &= 0.90(50\text{ ksi})\\ &= 45.0\text{ ksi} > 37.0\text{ ksi} \quad \textbf{o.k.}\end{aligned}$$

The beam has adequate flexural strength.

Comments:

As an alternative approach, the rotation of the beam can be determined using the "Flexural Analogy," which converts the applied torsion into a couple acting at the top and bottom flanges of the beam. The force at the top flange is assumed to be resisted by the slab. The force at the bottom flange is assumed to be resisted by weak-axis bending of the bottom half of the W24. The calculated deflection is then converted into an equivalent rotation about the top flange.

The moment of inertia of the bottom flange is,

$$I_{y\ FA} = \frac{I_y}{2} = \frac{82.5\ \text{in.}^4}{2} = 41.3\ \text{in.}^4$$

The equivalent force to apply to the bottom flange is,

$$F_{FA} = \frac{T}{d} = \frac{8.58\ \text{kip-ft}\left(12\ \text{in./ft}\right)}{23.9\ \text{in.}} = 4.31\ \text{kips}$$

The deflection of the bottom flange is,

$$\begin{aligned}\Delta_{bf\ FA} &= \frac{F_{FA} L_T^3}{48 E I_{y\ FA}} \\ &= \frac{\left(4.31\ \text{kips}\right)\left(10\ \text{ft}\right)^3 \left(12\ \text{in./ft}\right)^3}{48\left(29{,}000\ \text{ksi}\right)\left(41.3\ \text{in.}^4\right)} \\ &= 0.129\ \text{in.}\end{aligned}$$

Because the beam is forced to rotate about its top flange the designer can take advantage of the full beam depth when calculating θ. The equivalent rotation about the top flange using the flexural analogy is,

$$\begin{aligned}\theta_{FA} &= \sin^{-1}\left(\frac{\Delta_{bf\ FA}}{d}\right) \\ &= \sin^{-1}\left(\frac{0.129\ \text{in.}}{23.9\ \text{in.}}\right) \\ &= 0.310^\circ\end{aligned}$$

In this case the rotation predicted by the flexural analogy is approximately half of that predicted by analyzing the twist of the beam about its centroid. When the torsional length is less than about 15 ft, the flexural analogy is easier to calculate and provides a more accurate result.

The 0.337-in. deflection due to post composite load is approximately 80% of the total deflection. The rest of the deflection is due to twist of the beam between the roll beams. This example illustrates that the presence of roll beams will help to significantly reduce deflections at the precast panels. It also shows that under certain loading conditions one must still consider torsional deflections between roll beams.

When using the LRFD methodology with a commercially available computer program, it may be necessary to make two "runs" of the analysis—the first in LRFD-mode to obtain factored moments and the second in ASD-mode to obtain the effective elastic section modulus and post-composite deflections. To investigate the interaction of the stresses properly, only elastic stresses are used here. Using $M_r/\phi M_n$ in place of the first two terms in the stress interaction equation may be unconservative.

Example 6.6—Precast Panel Loads at Floor Opening

This example illustrates the design of a spandrel beam adjacent to an opening with eccentric cladding load. The example also shows how to analyze a built-up section consisting of a wide flange and supplemental side plate for torsion.

The 30-ft-long wide-flange spandrel beam shown in Figures 6-41 and 6-42 is adjacent to a stair opening over a portion of its span. The spandrel beam supports the floor framing shown in addition to 6-in.-thick precast concrete cladding panels weighing 75 psf. The panels bear on the top

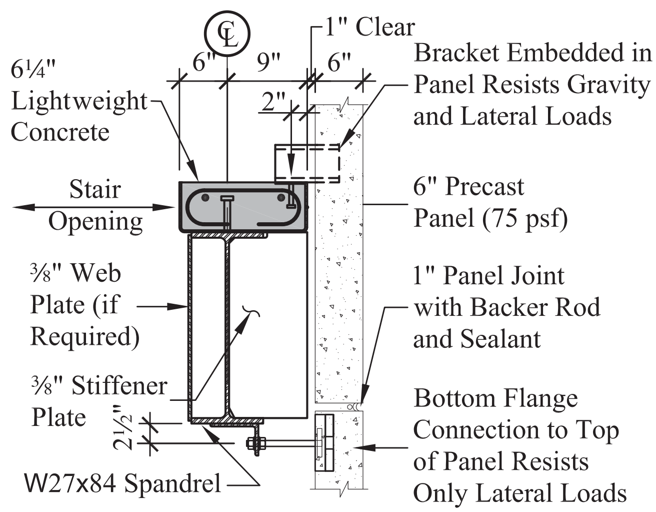


Fig. 6-41. Section of floor spandrel beam at opening with precast panel supported on slab.

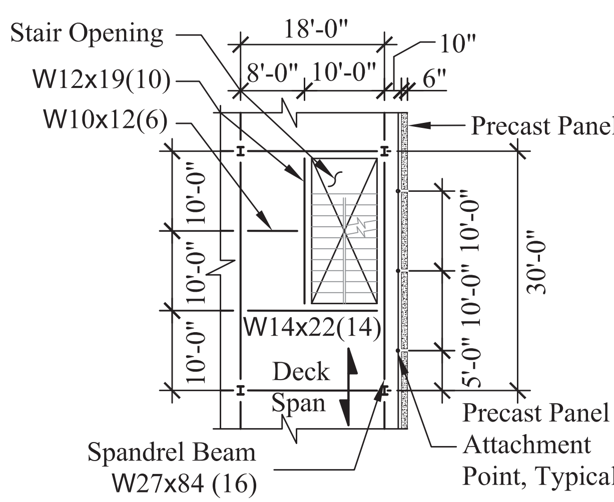


Fig. 6-42. Floor plan at spandrel beam.

of the slab 2 in. from the edge of the slab. The 6¼-in.-thick floor system consists of 3¼-in. lightweight concrete on 3-in. metal floor deck. The floor live load is 100 psf, and the superimposed dead load is 15 psf. The maximum vertical deflection due to superimposed loads is ½ in.

Determine the total vertical deflection at the support points for the precast panels considering torsion and the bare W27 section. If the deflection of the bare W27 section is excessive, add a second web to create a built-up box section to increase torsional stiffness. Check the strength and the horizontal displacement of the spandrel beam—for this case, use a horizontal deflection limit of $L/480$.

Given:

For this example assume the controlling load combination for strength is $1.2D + 1.6S + 0.8W$ ($D + S + W$ for deflection). The spandrel beam is a W27×84, while the W14×22 floor framing beam perpendicular to the spandrel provides torsional restraint for the spandrel. For the spandrel beam, L = 30 ft for flexure and L_T = 20 ft for torsion. The length of the W14×22 floor beam, L_{fb} = 10 ft. The story height, h = 12 ft.

The horizontal eccentricity between the centroid of the precast panel and the edge of the pour stop is resolved by a couple between adjacent floors. The pour stop is a ¼-in.-thick bent plate. The spandrel has web stiffeners at the precast attachment locations. Because of the opening, assume that the spandrel beam does not act compositely with the floor slab. The spandrel beam deflects a total of 0.48 in. due to live loads and superimposed dead loads, including the cladding.

The supports for the precast panels are spaced at s_p = 10 ft apart. The horizontal eccentricity between the point of the precast panel attachment and the centerline of the beam, e = 7 in. The horizontal eccentricity between the point of the precast panel attachment and centerline of the precast panels is e_p = 6 in. The vertical eccentricity between the bottom flange of the spandrel beam and the panel attachment point is e_r = 2½ in. Any additional contribution of the concrete slab strip to the torsional constant, J, of the section has been neglected.

The factored moment demand, M_u = 414 kip-ft. The available moment strength of the bare beam, considering a laterally unbraced length L_b = 20 ft, is $\phi_b M_n$ = 579 kip-ft, based on AISC Manual Table 3-10.

For the loads on the spandrel beam, see Figures 6-43, 6-44 and 6-45. The W27×84 spandrel beam has the following properties:

d = 26.7 in.
t_w = 0.460 in.
b_f = 10.0 in.
t_f = 0.640 in.
I_x = 2,850 in.^{4}
I_y = 106 in.^{4}
S_y = 21.2 in.^{3}
Z_y = 33.2 in.^{3}

J = 2.81 in.^{4}
a = 128 in.

The deflection due to superimposed dead load at midspan is Δ_{pcl} = 0.48 in.

The W14×22 floor framing beam has I_x = 199 in.^{4}

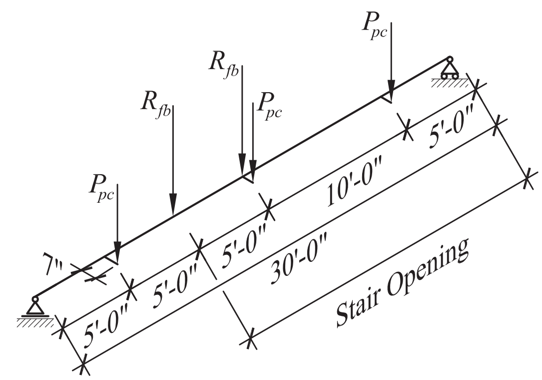


Fig. 6-43. Loads on spandrel beam.

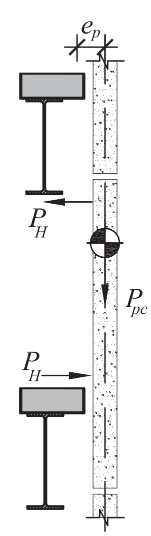


Fig. 6-44. Resolution of precast panel eccentricity.

Solution:

The service precast panel point load at each support point on the spandrel is,

$$\begin{aligned}P_{pc} &= \left(0.075 \text{ kip/ft}^2\right)hs_p\\ &= \left(0.075 \text{ kip/ft}^2\right)\left(12 \text{ ft}\right)\left(10 \text{ ft}\right)\\ &= 9.00 \text{ kips}\end{aligned}$$

The service wind point load on the spandrel beam at each precast panel support point location is,

$$\begin{aligned}P_w &= p_w s_p h\\ &= 0.030 \text{ kip/ft}^2\left(10 \text{ ft}\right)\left(12 \text{ ft}\right)\\ &= 3.60 \text{ kips}\end{aligned}$$

The service horizontal force on the spandrel beam due to eccentricity of panel with respect to pour stop is,

$$\begin{aligned}P_H &= \frac{P_{pc}e_p}{h-d-e_r}\\ &= \frac{9.00 \text{ kips}\left(6 \text{ in.}\right)}{12 \text{ ft}\left(12 \text{ in./ft}\right)-26.7 \text{ in.}-2\tfrac{1}{2} \text{ in.}}\\ &= 0.470 \text{ kip}\end{aligned}$$

The factored horizontal force on the spandrel beam due to the eccentricity of the panel with respect to the pour stop is,

$$\begin{aligned}P_{uH} &= 1.2\left(P_H\right)\\ &= 1.2\left(0.470 \text{ kip}\right)\\ &= 0.564 \text{ kip}\end{aligned}$$

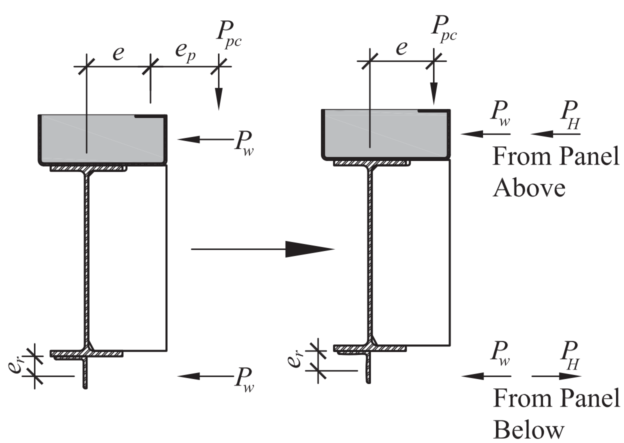


Fig. 6-45. Vertical and lateral forces on spandrel beam.

The deflection of the W27 section will be evaluated without the slab to determine its contribution to the deflection at the tip of the bent plate pour stop considering torsion. For the purpose of this analysis, the perpendicular floor framing beam and columns will be considered to restrain the torsion of the spandrel beam. Using AISC Design Guide No. 9 (Seaburg and Carter, 1997), the maximum angle of twist can be determined. Unlike the previous example; however, in this example torsion is calculated with respect to the centroid of the spandrel beam since the top flange is not restrained laterally by the floor slab.

Note that because of the horizontal eccentricity, e_p, between the centroid of the panel and the point of vertical support, the weight of the panel creates a couple between the flanges of the spandrel that opposes the torsion caused by P_{pc}. The service torsional moment due to the precast load at the point of attachment is,

$$\begin{aligned}T_{pc} &= P_{pc}e - 2P_H\left(\frac{d}{2}+e_r\right)\\ &= \frac{9.00 \text{ kips}\left(7 \text{ in.}\right)-2\left(0.470 \text{ kip}\right)\left(\dfrac{26.7 \text{ in.}}{2}+2\tfrac{1}{2} \text{ in.}\right)}{12 \text{ in./ft}}\\ &= 4.01 \text{ kip-ft}\end{aligned}$$

Unlike the previous example in which the wind loads caused twist on the bottom flange of the spandrel, here the wind loads are applied equally to both the top and bottom flange of the beam. Although there is no resulting twist, the beam is loaded in weak-axis flexure between the column and the W14×22 floor framing beam. The total service torsional moment at the point of the precast attachment is,

$$\begin{aligned}T &= T_{pc}\\ &= 4.01 \text{ kip-ft}\end{aligned}$$

To obtain the service flexural moment applied to the end of the W14×22 floor-framing beam, T from the span adjacent to the stair is added to $T/2$ from the span adjacent to the stair. The service moment applied to the end of the floor framing beam is,

$$\begin{aligned}M_{fb} &= T+\frac{T}{2}\\ &= 4.01 \text{ kip-ft}+\frac{4.01 \text{ kip-ft}}{2}\\ &= 6.02 \text{ kip-ft}\end{aligned}$$

The rotation at the end of the floor framing beam is (see Figure 6-46),

$$\theta_{fb} = \frac{M_{fb} L_{fb}}{3EI_{xfb}}$$
$$= \frac{6.02 \text{ kip-ft}\left(10 \text{ ft}\right)\left(12 \text{ in./ft}\right)^2}{3\left(29{,}000 \text{ ksi}\right)\left(199 \text{ in.}^4\right)}$$
$$= 0.000501 \text{ rad. (or } 0.0287°)$$

This rotation is very small and has been neglected in subsequent calculations.

The service vertical reaction on the spandrel due to the end moment on the floor framing beam is,

$$R_{fb} = \frac{M_{fb}}{L_{fb}}$$
$$= \frac{6.02 \text{ kip-ft}}{10 \text{ ft}}$$
$$= 0.602 \text{ kip}$$

This reaction is small and can be ignored for calculations.

Assuming that the spandrel beam is restrained for torsion at each end of the floor opening, the spandrel beam deflection can be calculated as follows using AISC Design Guide No. 9, Appendix B, case 3. The torsional constant, J, for the bare steel beam is used, neglecting any additional contribution of the concrete slab strip. Note that this case does not provide values for two torques applied at the one-quarter points of the span. Therefore, superposition will be used to determine the rotation at the midspan of the opening. Interpolating between the charts for $\alpha = 0.1$ and $\alpha = 0.3$, the rotation at midspan due to a single point load at $\alpha = 0.25$ is,

$$\frac{L_T}{a} = \frac{20 \text{ ft}\left(12\text{in./ft}\right)}{128 \text{ in.}} = 1.88$$

$$\theta\left(\frac{GJ}{TL_T}\right) = 0.040 \text{ rad.}$$

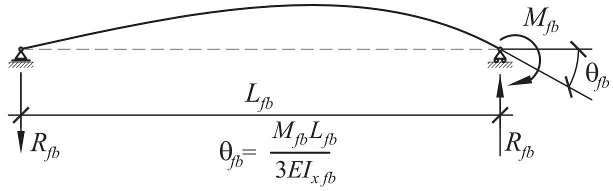


Fig. 6-46. Rotation of roll beam.

$$\theta = \frac{(0.040 \text{ rad.})TL_T}{GJ}$$
$$= \frac{\left(0.040 \text{ rad.}\right)\left(4.01 \text{ kip-ft}\right)\left(20 \text{ ft}\right)\left(12 \text{ in./ft}\right)^2}{\left(11{,}200 \text{ ksi}\right)\left(2.81 \text{ in.}^4\right)}$$
$$= 0.0147 \text{ rad. (or } 0.841°)$$

To obtain the total rotation at midspan due to two point loads superimposed,

$$\theta_{total} = 2\theta$$
$$= 2\left(0.841°\right)$$
$$= 1.68°$$

The vertical deflection at the tip of the pour stop due to rigid body rotation is,

$$\Delta_{vrb} = e \sin\left(\theta_{total}\right)$$
$$= \left(7 \text{ in.}\right)\sin(1.68°)$$
$$= 0.205 \text{ in.}$$

The total vertical deflection at the center of the opening is,

$$\Delta_{total} = \Delta_{vrb} + \Delta_{pcl}$$
$$= 0.205 \text{ in.} + 0.48 \text{ in.}$$
$$= 0.685 \text{ in.} \geq \tfrac{1}{2} \text{ in.} \quad \textbf{n.g.}$$

Because the deflection exceeds the established limit, consider adding a second web to the W27×84 to create a box section, which will have substantially more torsional stiffness. Try a ⅜-in.-thick plate to the side of the beam toward the opening.

The torsional constant, J, of the closed section, neglecting the outer flanges of the W27×84, can be calculated using AISC Design Guide No. 9, Table 3.1. The distance from the centerline of the spandrel beam to the centerline of the plate is,

$$b_b = \frac{b_f}{2} - \frac{t_w}{2} + \frac{t_p}{2}$$
$$= \frac{10.0 \text{ in.}}{2} - \frac{0.460 \text{ in.}}{2} + \frac{3/8 \text{ in.}}{2}$$
$$= 4.96 \text{ in.}$$

The centerline-to-centerline height of the box section is,

$$h_b = d - t_f$$
$$= 26.7 \text{ in.} - 0.640 \text{ in.}$$
$$= 26.1 \text{ in.}$$

The least thickness of the web and plate can be used to determine J.

$$t_{wb} = \min(t_p,\ t_w)$$
$$= \min(3/8 \text{ in.},\ 0.460 \text{ in.})$$
$$= 3/8 \text{ in.}$$

The torsional constant, J, of the box section is,

$$J = \frac{2t_f t_{wb} b_b^2 h_b^2}{b_b t_{wb} + h_b t_f}$$

$$= \frac{2(0.640 \text{ in.})(\tfrac{3}{8} \text{ in.})(4.96 \text{ in.})^2 (26.1 \text{ in.})^2}{4.96 \text{ in.}(\tfrac{3}{8} \text{ in.}) + (26.1 \text{ in.})(0.640 \text{ in.})}$$

$$= 433 \text{ in.}^4$$

The maximum rotation of the box section will occur at the point of the precast attachment. The rotation of the beam will be constant between the points of applied torque. The torsional rotation of the box section is,

$$\theta_b = \frac{T\left(\frac{L_T}{4}\right)}{GJ}$$

$$= \frac{4.01 \text{ kip-ft}\left(\frac{20 \text{ ft}}{4}\right)(12 \text{ ft/in.})^2}{11{,}200 \text{ ksi } (433 \text{ in.}^4)}$$

$$= 0.000595 \text{ rad. (or } 0.0341°)$$

The vertical deflection at the tip of the pour stop due to the rigid body rotation is,

$$\Delta_{vrb} = e \sin(\theta_b)$$

$$= (7 \text{ in.})\sin(0.0341°)$$

$$= 0.00417 \text{ in.}$$

Neglecting the contribution of the second web plate to the flexural stiffness of the section, the total deflection at the center of the slab opening is,

$$\Delta_{total} = \Delta_{vrb} + \Delta_{pcl}$$

$$= 0.00417 \text{ in.} + 0.48 \text{ in.}$$

$$= 0.484 \text{ in.} \le \tfrac{1}{2} \text{ in.} \quad \textbf{o.k.}$$

To check the lateral (out-of-plane) deflection of the spandrel beam, the spandrel can be assumed as pinned for out-of-plane flexure at the ends of the opening. The weak-axis moment of inertia of the built-up section is,

$$I_{yb} = 296 \text{ in.}^4$$

The deflection limit was selected as,

$$\frac{L_T}{480} = \frac{20 \text{ ft (12 in./ft)}}{480}$$

$$= \tfrac{1}{2} \text{ in.}$$

Using the AISC Manual Table 3-23, case 9, with the concentrated loads at the one-quarter points and simple supports, the out-of-plane deflection based on the horizontal deflection at midspan is,

$$\Delta_h = \frac{11 P_w L_T^3}{384 E I_{yb}}$$

$$= \frac{11(3.60 \text{ kips})(20 \text{ ft})^3 (12 \text{ in./ft})^3}{384(29{,}000 \text{ ksi})(296 \text{ in.}^4)}$$

$$= 0.166 \text{ in.} \le L_T/480 \quad \textbf{o.k.}$$

This calculation assumes that the beam is simply supported at each end of the floor opening. Because the beam is continuous past the W14 floor framing beam at the edge of the opening, the calculation is conservative. If a less conservative solution is desired, the continuity can be considered.

To check the strength of the spandrel beam, with the second web plate the W27×84 can be considered as a box section, neglecting warping normal stresses as described in AISC Design Guide No. 9. For simplicity in checking weak-axis flexure, the spandrel beam will be assumed as simply supported between the column and the W14×22 floor framing beam. The ⅜-in.-thick web plate will also be neglected for strength. The factored wind point load at each precast panel support point on the spandrel is,

$$P_{uw} = 0.8P_w$$

$$= 0.8(3.60 \text{ kips})$$

$$= 2.88 \text{ kips}$$

The required weak-axis flexural strength on the spandrel beam between supports is,

$$M_{uy} = \frac{P_{uw} L_T}{3}$$

$$= \frac{2.88 \text{ kips}(20 \text{ ft})}{3}$$

$$= 19.2 \text{ kip-ft}$$

The required torsional strength is,

$$T_u = 1.2T$$

$$= 1.2(4.01 \text{ kip-ft})$$

$$= 4.81 \text{ kip-ft}$$

The available torsional strength of the built-up box section can be calculated assuming that the section is comprised entirely of ASTM A572 Grade 50 plate material (F_y = 50 ksi) except when calculating slenderness ratios (a conservative lower bound). The box width-thickness ratio is as follows,

$$\frac{h}{t_w} = \frac{26.1 \text{ in.}}{0.375 \text{ in.}} = 69.6$$

The selection of the appropriate torsional strength equation is based on the following limiting wall slenderness ratios for box sections (from AISC Specification Section H3.1) (AISC, 2005b), based on $F_y = 50$ ksi for the W27,

$$\lambda_p = 2.45\sqrt{\frac{E}{F_y}} = 2.45\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} = 59.0$$

$$\lambda_r = 3.07\sqrt{\frac{E}{F_y}} = 3.07\sqrt{\frac{29{,}000\text{ ksi}}{50\text{ ksi}}} = 73.9$$

Because $59.0 < h/t_w < 73.9$, the critical stress for torsion is,

$$\begin{aligned}F_{cr} &= \frac{0.6F_y\left(2.45\sqrt{\frac{E}{F_y}}\right)}{\left(h/t_w\right)}\\ &= \frac{0.6\left(50\text{ ksi}\right)\left(59.0\right)}{\left(69.6\right)}\\ &= 25.4\text{ ksi}\end{aligned}$$

The box section torsional shear constant, per the AISC Specification Section H3.1 User Note, is,

$$\begin{aligned}C &= 2b_bh_bt_{wb} - 4.5\left(4-\pi\right)t_{wb}^3\\ &= 2\left(4.96\text{ in.}\right)\left(26.1\text{ in.}\right)\left(\tfrac{3}{8}\text{ in.}\right) - 4.5\left(4-\pi\right)\left(\tfrac{3}{8}\text{ in.}\right)^3\\ &= 96.9\text{ in.}^3\end{aligned}$$

The available torsional strength is,

$$\phi_T = 0.90$$

$$\begin{aligned}\phi_T T_n &= \phi_T F_{cr} C\\ &= \frac{0.90\left(25.4\text{ ksi}\right)\left(96.9\text{ in.}^3\right)}{12\text{ in./ft}}\\ &= 185\text{ kip-ft}\end{aligned}$$

The required torsional strength to design torsional strength ratio is,

$$\frac{T_u}{\phi_T T_n} = \frac{4.81\text{ kip-ft}}{185\text{ kip-ft}} = 0.0260$$

Because this ratio is less than 20 percent, AISC Specification Section H3.2 permits the use of the equations in AISC Specification Section H1 to evaluate the interaction of flexural forces. For simplicity, it is conservative to neglect the added flexural strength of the second web. Refer to AISC Specification Section F6.1, noting that the W27×84 is not susceptible to flange local buckling. The available weak-axis flexural strength is calculated as,

$$\begin{aligned}\phi_b M_{ny} &= \phi_b F_y Z_y \le \phi_b 1.6 F_y S_y\\ &= \frac{\left(0.90\right)\left(50\text{ ksi}\right)\left(33.2\text{ in.}^3\right)}{12\text{ in./ft}}\\ &\le \frac{0.90\left(1.6\right)\left(50\text{ ksi}\right)\left(21.2\text{ in.}^3\right)}{12\text{ in./ft}}\\ &= 125\text{ kip-ft} \le 127\text{ kip-ft}\\ &= 125\text{ kip-ft}\end{aligned}$$

If a more accurate calculation for the weak-axis flexural strength is necessary, the pour stop plate and ⅜-in.-thick web can be included in the available strength calculations. Because the spandrel has no axial load, AISC Specification Equation H1-1b applies.

$$\begin{aligned}\frac{M_{ux}}{\phi_b M_{nx}} + \frac{M_{uy}}{\phi_b M_{ny}} &= \frac{414\text{ kip-ft}}{579\text{ kip-ft}} + \frac{19.2\text{ kip-ft}}{125\text{ kip-ft}}\\ &= 0.870 \le 1.0 \quad \textbf{o.k.}\end{aligned}$$

Even considering the conservative nature of this calculation, the section has adequate strength.

Comments:

This example is detailed with a strip of slab on the spandrel beam that has not been counted in the composite action of the slab. The strip of slab is provided to maintain the same edge condition along the entire building. As an alternative solution, it is also permissible to eliminate the slab on the portion of the spandrel beam adjacent to the opening. In this case, the cladding attachment detail would be modified.

Additionally, in this example, the precast cladding panels are supported for gravity loads at 10 ft on center along the spandrel beam. This type of panel arrangement imposes a substantial penalty on the spandrel beam weight because a relatively large moment of inertia is required, particularly because the beam is non-composite. If acceptable to the architect, the weight of the spandrel would be greatly reduced if the precast panels could span from column to column, as the spandrel beam would not be subject to special deflection criteria.

A complete design of this spandrel would include a design for the welds between the second web plate and the flanges, as well as designs for connections at the ends of the spandrel to resist torsion and vertical shear. This is left as an exercise for the reader.

Chapter 7
Masonry Cavity Wall Systems with Concrete Masonry Unit or Metal Stud Back-Up

7.1 GENERAL DESCRIPTION OF MASONRY CAVITY WALL SYSTEMS

The common components of a typical masonry cavity wall system include (working from the outside in):

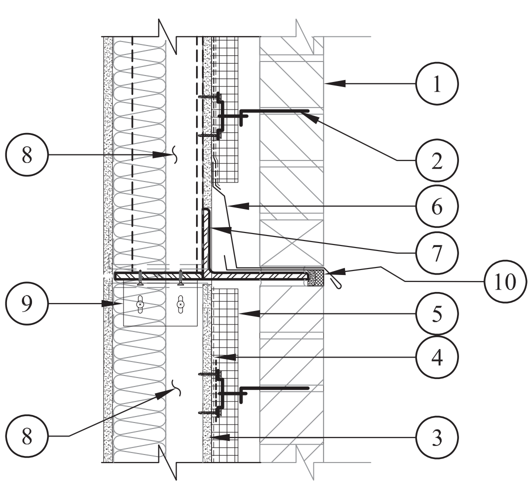
- Brick or stone veneer, which is usually of 4 in. nominal thickness but can range in thickness from 1½ in. to 6 in.
- Air cavity, which is usually a minimum of 2 in. clear.
- Insulation, the thickness of which varies with the project location and energy demands.
- Waterproofing membrane, which is sometimes also the air barrier.
- Structural back-up to support the masonry for out-of-plane loads, often composed of concrete block masonry or gypsum sheathing on cold-formed metal studs.

The masonry assemblage is usually supported by continuous shelf angles, which are supported by the primary building structure. A shelf angle support is often provided at each story, but occasionally at every other story. For low-rise buildings, the veneer is sometimes supported from the foundation without support at the floor levels.

When veneer is supported by a floor, properly designed movement joints below the shelf angle prevent differential floor deflection of one story from transferring loads to the veneer below, and control movement from moisture and thermal volume change. The veneers are nonstructural elements. The shelf angle is also the location of through-wall flashing that catches any water that has entered the cavity and diverts it back out of the cavity. Figure 7-1 illustrates a typical masonry cavity wall system.

The veneer is “tied” to the back-up with steel anchors to provide out-of-plane support. The size and spacing of the anchors depends on the project wind and seismic loads, the veneer type, and the support conditions. Generally, the anchors are arranged such that there is one every 2 to 4 ft^2 of veneer. The anchors are called brick ties in brick veneer systems and stone anchors in stone veneer systems.

In addition to the horizontal movement joints provided below each shelf angle, vertical control joints are provided for volume change and other movements. The vertical control



1. Veneer.
2. Veneer anchor.
3. Exterior sheathing, usually gypsum based.
4. Water barrier.
5. Insulation.
6. Through wall flashing.
7. Shelf angle.
8. Back-up wall (metal stud shown).
9. Top of back-up wall connection allows vertical movement between portion of wall above soft joint and portion below, plus in-plane movement.
10. Soft joint under shelf angle.

Fig. 7-1. Typical masonry cavity wall system.

joints are located at or near corners and spaced throughout the field of the wall depending on factors such as the veneer material, the location of openings, and the aspect ratio of the panel bounded by the vertical joints and the horizontal joints. A common spacing of vertical joints is approximately 20 ft. Figure 7-2 shows the movement joints in an elevation of a typical masonry veneer system.

7.2 STRATEGIES FOR SUPPORT OF MASONRY CAVITY WALL SYSTEMS

A strategy for supporting the masonry cavity wall system starts with the architect's decision for the location of the horizontal movement joints. If the architect elects to place the movement joint at an elevation that aligns with the head of the windows at each story, the shelf angle will most likely be hung from the spandrel beam because the head of the windows is usually below the bottom flange of the spandrel. If the shelf angle elevation aligns with the head of punched window openings (in other words not strip windows), it is usually most common to support the back-up on the edge of the slab. Figure 7-3 depicts a wall section from such a case with punched windows.

When the shelf angle is at the window head, a hanger assembly usually supports the veneer and the back-up that is above the window head and below the slab. Therefore, the veneer and back-up above the window head are supported by—and move with—the spandrel beam and floor framing of the floor above. In addition, the hanger assembly must provide lateral out-of-plane support for the top of the back-up below. This lateral force below the spandrel, combined with the vertical force from the veneer and back-up eccentric to the spandrel, applies torsion to the spandrel that is generally resolved with kickers and/or floor or roll beams as described in previous chapters.

If the architecture incorporates strip windows, the shelf angle should be at the head of the strip window to support the veneer above the strip window and the out-of-plane loads from the window head. The weight of the strip window is supported on either the back-up or the veneer below it. With strip window conditions, designers often run the back-up by the edge of the slab. In this way, the back-up can resist out-of-plane loads by cantilevering past the edge of the slab up to the underside of the strip window. This is also a convenient way to form parapets. Figure 7-4 depicts such a case with strip windows.

An alternative to running the back-up by the edge of slab at strip windows and parapets is to cantilever the back-up from the top of slab. However, this requires a moment connection to the slab, and requires the slab to have adequate flexural strength for both positive and negative out-of-plane loads.

If the architect elects to place the shelf angle at or near the level of the slab, it may be possible to support the shelf angle directly from the slab edge or from stiffener plates attached to the spandrel. The back-up from the story below can extend to the underside of the slab if it overhangs the spandrel sufficiently. Figure 7-5 shows such a wall section.

In this case, punched window openings in the veneer will require "loose lintels" to support the veneer over the opening. These lintels are supported by the veneer itself and move with that panel of veneer. It is important not to attach loose lintels to the back-up unless vertical control joints are provided at each end of the lintel. Otherwise the relative movement between the veneer supported on the lintel, which is supported from the back-up, and the adjacent veneer supported from the shelf angle on the floor below may cause cracking in the veneer at the ends of the lintel.

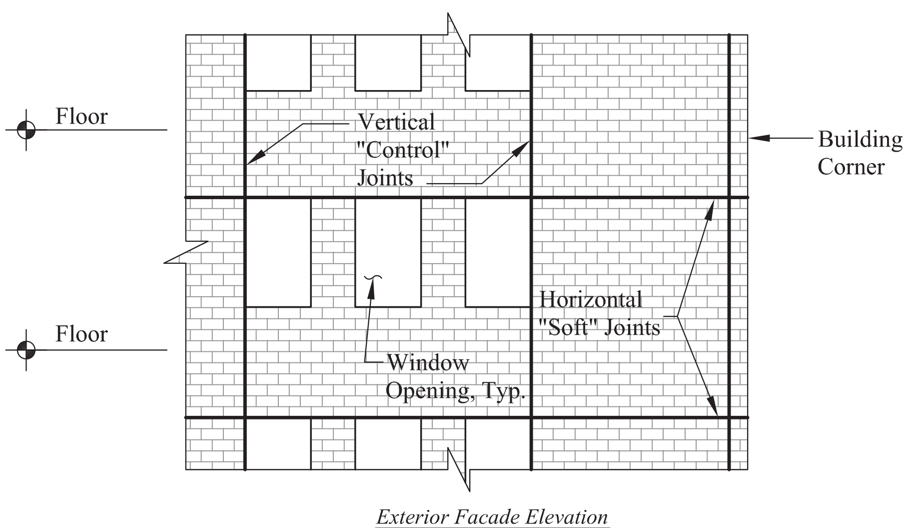
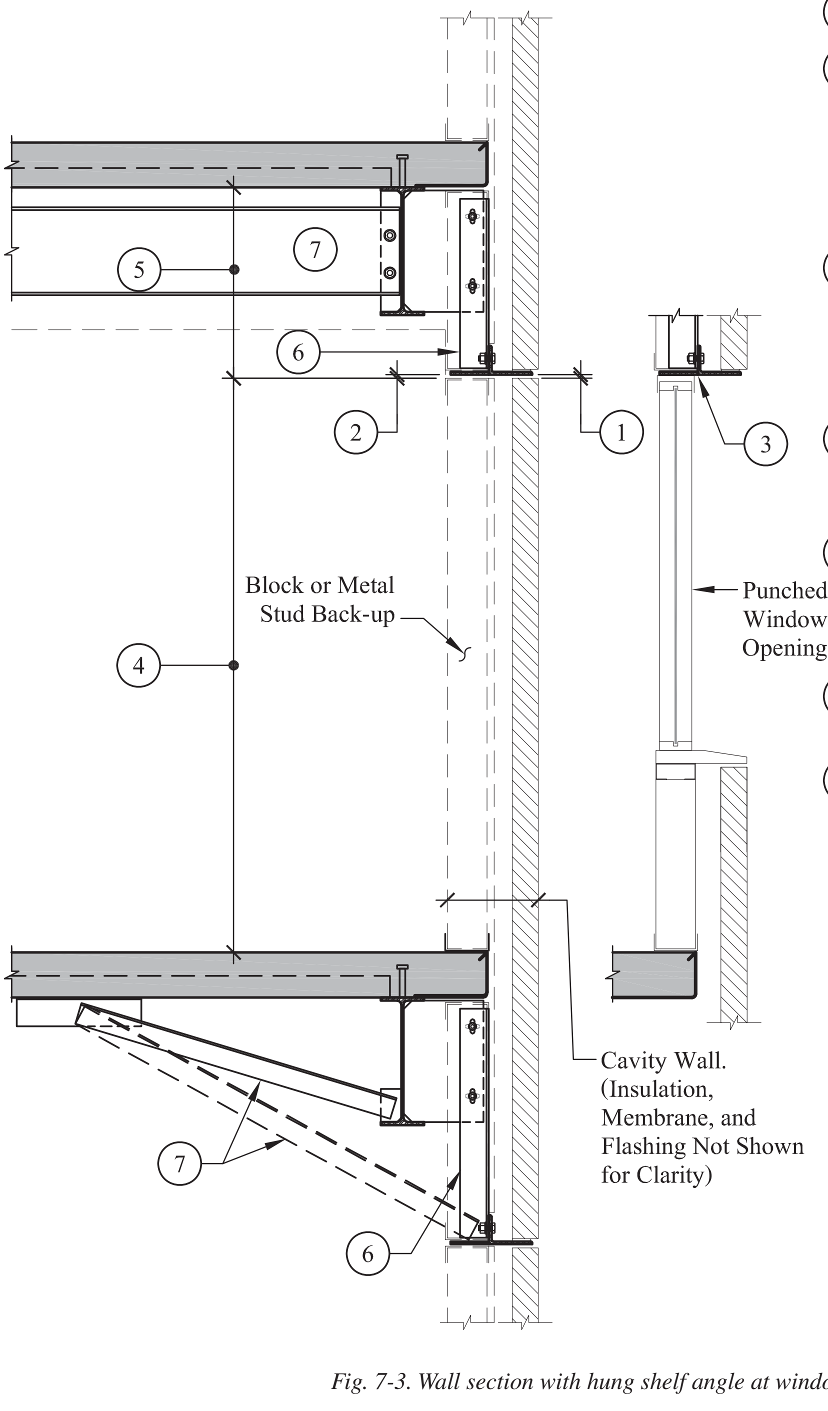


Fig. 7-2. Elevation of masonry veneer with movement joints.



NOTES:

1 Soft joint below shelf angle.

2 Movement joint between shelf angle assembly and back-up wall below. The connection to underside of shelf angle assembly provides out-of-plane lateral support to the back-up wall.

3 Movement joint between shelf angle assembly and window head. The window head connection provides out-of-plane support of the window.

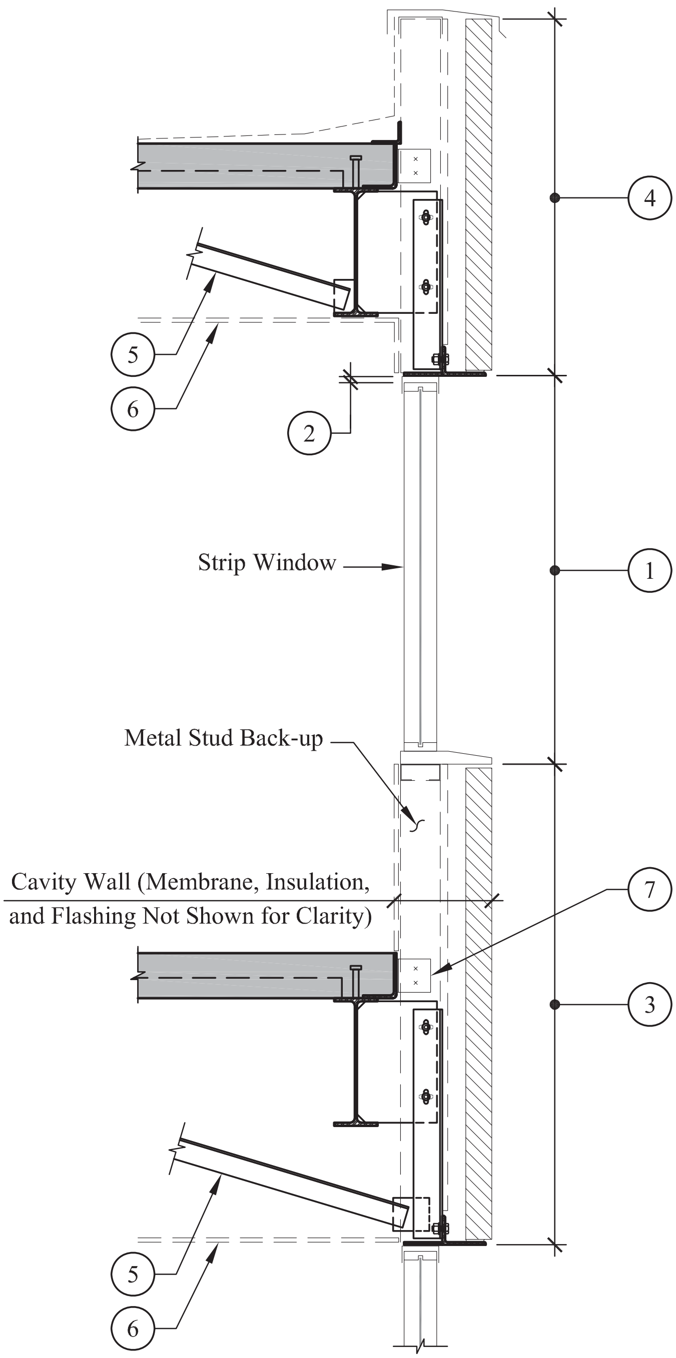
4 Back-up wall of block or metal studs below movement joint is supported on floor slab.

5 Back-up wall above movement joint is supported on shelf angle assembly and anchored to underside of floor slab.

6 Hangers from plate brackets support the shelf angle assembly.

7 Roll beams or kickers can resist the twist of the spandrel beam.

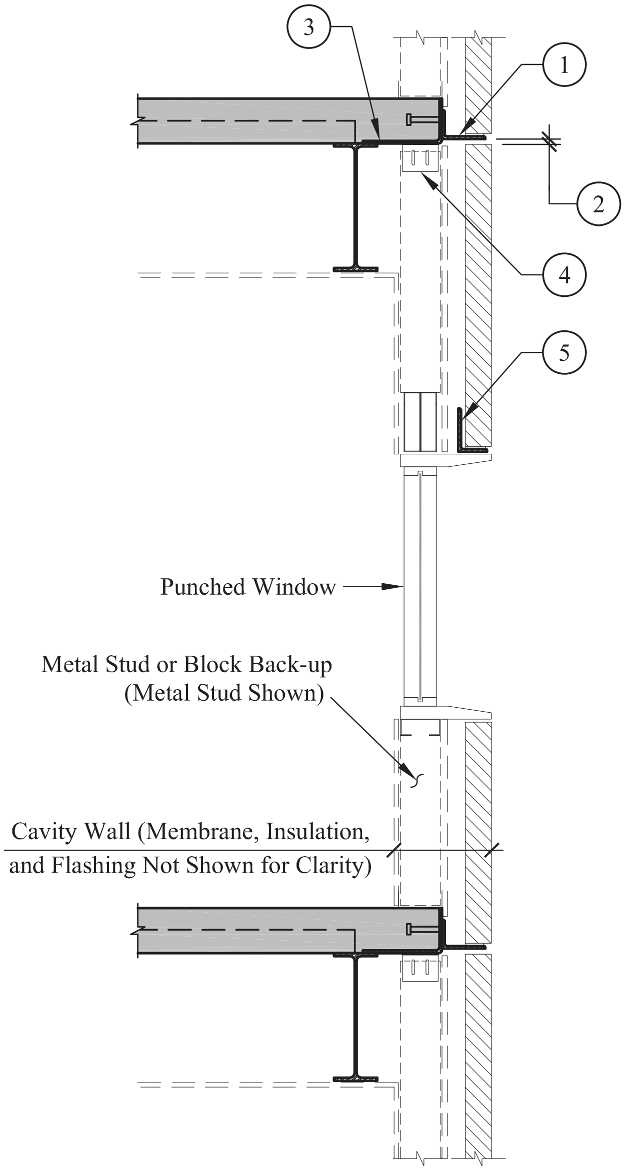
Fig. 7-3. Wall section with hung shelf angle at window head.



NOTES:

1. Strip window. The shelf angle is at the window head.
2. Movement joint between shelf angle assembly and window head. The window head connection provides out-of-plane support of the window.
3. Metal stud back-up wall is supported on the hung shelf angle assembly. Studs are connected to the edge of the slab and cantilever up to provide vertical and out-of-plane support at the sill of the strip window.
4. At the roof, the metal studs cantilever up past the slab edge to form the parapet.
5. Kickers or roll beams can resist the twist of the spandrel beam.
6. The finish ceiling location may dictate the location of the kickers.
7. Lateral tie to slab so studs can cantilever past the edge of slab up to sill of window for out-of-plane support of window.

Fig. 7-4. Wall section with hung shelf angle and strip windows (back-up runs past slab edge).



NOTES:

1. Shelf angle attached to slab edge.
2. Soft joint under shelf angle.
3. Bent plate forms edge of slab and provides the means to connect the shelf angle to the slab.
4. The back-up is connected to the underside of the floor slab for out-of-plane restraint only and allows vertical and in-plane movement.
5. Loose lintel over punched opening. The lintel supports the veneer over the window and bears on the veneer. The lintel is not anchored to the back-up.

Fig. 7-5. Wall section with shelf angle supported at slab.

7.3 PARAMETERS AFFECTING DESIGN OF MASONRY CAVITY WALL SUPPORTS

Parameters affecting the design of masonry cavity wall systems include:

- Architectural decisions.
- Dimensional considerations.
- Magnitude of seismic forces.
- Field adjustability for tolerances and clearances.
- Relative movements between the primary structure and the wall system.
- Durability considerations.

Of these, the architectural and dimensional considerations are the parameters that the design team has the most control over. Thus it is with these parameters that the design team can have the largest impact on cost.

7.3.1 Architectural Decisions That Impact the Design of the Masonry Cavity Wall

Architectural decisions dictate the strategy for supporting the wall system as discussed in the preceding section. Architectural decisions that affect the support and attachment of the wall system will now be discussed in more detail.

Location of the Horizontal Soft Joints (and Shelf Angles That Support the Masonry Veneer)—As discussed before, the elevation of the soft joint relative to the floor slab elevation drives the designer's decision on how to support the wall. It is usually economical if the soft joint and shelf angle are located at the heads of the windows below the subject floor and remain at a constant elevation around the perimeter of the building at the floor (or at least constant for those portions of the perimeter that are masonry cavity walls). The closer the heads of the windows are to the underside of the spandrel beams, the less distance the shelf angle needs to be hung, and this generally means less cost. If the shelf angle changes elevation, the veneer should have a vertical control joint.

If the horizontal soft joint is located at an elevation within the depth of the spandrel beam, the shelf angle can be attached without hangers. If the shelf angle elevation is near the slab edge, it may be possible to fasten it directly to the slab, again avoiding hangers. However, for these two cases, the location of the windows may require loose lintels in the veneer, the cost of which may offset the savings of not having hangers.

Story Height—The story height directly affects the weight of the veneer and back-up supported by the shelf angle. The story height also affects the magnitude of the out-of-plane forces that must be transferred from the back-up to the primary structure. The difference between the story height and the window head height affects the length of hangers unless the design uses loose lintels over the windows. Designers may elect to use loose lintels for large story heights with low window heads.

Strip Windows—Continuous bands, or very wide bands, of windows require the shelf angle to be at the head of the windows to support the veneer above, regardless of the story height.

Finished Ceiling Elevation—The elevation of the finished ceiling relative to the window head will affect the opportunity for adding kickers.

Mechanical Systems—Mechanical system distribution near the perimeter of the building also affects the opportunity for adding kickers.

Window Head Detail—If the shelf angle is located just above the window head location, the window head will typically be anchored to the shelf angle or another part of the hanger assembly for out-of-plane forces. The architectural detail must include the anchorage of the window, flashing and other waterproofing measures, continuity of air barriers, thermal breaks, and other architectural finish features, such as ceilings, window shade pockets, and/or trims. All of these features influence the geometry and location of the wall support steel at this detail. It is best if the geometry and location for the support steel chosen for the window head also works for the soft joint detail between windows so that the support steel detail can be constant around the perimeter of the building. Sample details are shown in Figure 7-6.

Location of Vertical Control Joints in Veneer—The location of the vertical control joints in the veneer depends on the material and geometry of the veneer, and its openings. Joints in the shelf angle do not need to align with the vertical control joints in the veneer unless there is a change in shelf angle elevation, or, if the designer expects differential vertical deflection of the shelf angles across the joint due to a significant change in the flexibility of the shelf angle on either side of the joint.

7.3.2 Dimensional Considerations

The elevation of the shelf angle relative to the floor and window head elevations was discussed in the preceding section. Following are additional dimensional considerations.

Horizontal Leg of Shelf Angle—The insulation thickness, cavity space, and veneer thickness comprise the plan dimension from the exterior face of veneer to the face of waterproofing on the back-up surface. Ideally, the horizontal leg of the shelf angle is approximately equal to this dimension less the amount that the veneer overhangs the tip of the leg.

However, with modern energy codes increasing the thickness of insulation required in the cavity, it may become impractical to have the shelf this wide. When the vertical leg of the shelf angle is outward from the face of the back-up, a structural spacer can be used between the back of the angle and the hanger so that the outside face of the hanger can be behind the waterproofing membrane. See Figure 7-6. At a minimum, two-thirds of the width of the masonry veneer should bear on the shelf angle.

It is best practice to coordinate the dimension of the horizontal leg of the shelf angle so that a rolled angle shape fits. Trimming the leg of a standard-shaped angle is not economical. Using a bent plate is generally not practical because the tolerances of the bending are not adequate, the bend radius for the required thickness is relatively large, and the fabrication is more costly.

Thickness of the Back-up Wall—The thickness of the back-up wall may affect the designer's decisions about the location of the slab edge, the centerline of the spandrel beam, and the centerline of the exterior columns. Considerations include whether the back-up runs past the edge of the slab or past the spandrel beam to the underside of the slab, and whether the back-up runs outside the columns or the columns interrupt the back-up.

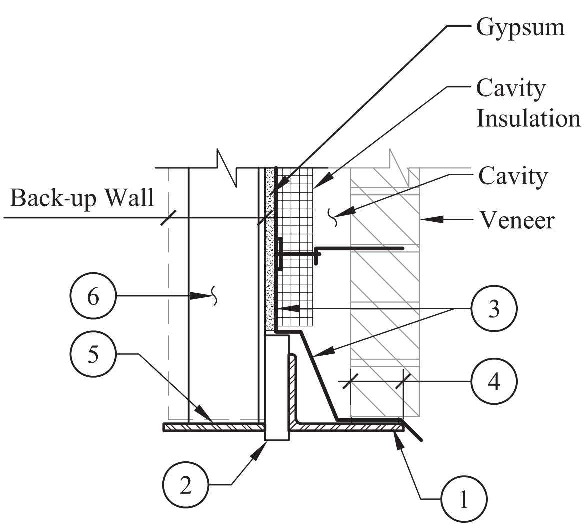
Relative Geometry of Slab Edge, Spandrel Centerline, and Column Centerline—This geometry must come together to balance competing requirements. For example, increasing the slab edge overhang from the spandrel and column centerlines may facilitate the back-up wall cleanly running past the column. However, this may also increase the eccentricity of the load. Holding the spandrel centerline back from the slab edge increases space for clearances and adjustability of the shelf angle assembly but, again, increases eccentricity.

7.3.3 Field Adjustability

Given the fabrication and erection tolerances of the steel frame, the support details for the masonry cavity walls require field adjustments. The magnitude of the necessary adjustments depends on the number of stories and the span lengths as discussed in Chapter 4. It is not uncommon for project conditions to necessitate the ability to adjust the location of the tip of the shelf angle to within ±⅛ in. of theoretical vertical and horizontal. Suggested adjustments include:

- Means to adjust the slab edge in or out relative to the spandrel beam.
- Means to adjust the location of the back-up wall in or out relative to the slab edge.
- Means to adjust the location of the shelf angle both vertically and horizontally.

Most projects have the final shelf angle location set by the steel erector as a follow-up operation after erecting and plumbing the frame. The use of erection bolts in slotted



NOTES:

1. Shelf angle made from standard rolled angle shape.
2. Structural spacer may be required for projects with thick insulation requirements in the cavity or with thick veneer in order to keep standard rolled angle as shelf.
3. Line of membrane and flashing.
4. A minimum of ⅔ of the veneer should bear on the shelf angle.
5. Continuous plate to support back-up wall above.
6. Structural hangers behind membrane and within the thickness of the back-up wall.

Fig. 7-6. Shelf angle with structural spacer to extend shelf under veneer.

holes with field welding is the most common mechanism for achieving the necessary field adjustments.

7.3.4 Movement Requirements

Both vertical and in-plane movements will be discussed.

Vertical Movements—The design of the wall support and attachments must account for the vertical deflections of the supporting steel and accommodate the vertical movements of the veneer due to volume change.

- Stiffness of spandrel beam and supporting structure: The designer must provide enough stiffness to mitigate cracking of the veneer due to support deflection. Limiting the spandrel beam and associated support deflections to less than the span divided by 600 is usually sufficient for most stone and brick veneers. ACI 530-05/ASCE 5-05/TMS 402-05 limits deflections from dead and live loads to the span divided by 600 or 0.3 in. However, this criterion usually does not control the design for typical spans of modern steel structures with spans of 25 to 40 ft. Instead, the spandrel stiffness will usually be limited by the acceptable horizontal soft joint size. Note that there is a tradeoff between width of joints and required stiffness of spandrel beams.

- Design movement of horizontal soft joints: Horizontal soft joints must accommodate the vertical movement due to the relative deflections of upper and lower floor structures, plus the movement due to thermal and moisture volume changes in the veneer. The sum of the movement accommodated by the soft joint will be approximately 25 to 50 percent of the soft-joint size, depending on sealant materials and sealant bond, to account for sealant compressibility and extensibility. Thus, the stiffness of the spandrel and shelf angle supports is often controlled by the acceptable soft-joint size. This is especially true for brick veneers as the soft joints must accommodate the long term expansion of the brick due to absorption of moisture. Example 7.1 illustrates the design concepts of selecting the horizontal soft-joint size.

In-Plane Drift of the Steel Frame—The support and attachment details must allow the primary structure to undergo its design drift without restraint from the non-structural cavity wall. As the structure undergoes drift, the upper floor is moving laterally relative to the lower floor. The common strategy of accommodating this movement is to anchor each story of wall to the lower floor where its weight is supported, and provide only out-of-plane lateral restraint at the top of the wall by connection to the upper floor. The connection at the top of the wall allows vertical movement and movement in the plane of the wall. The story of wall should not be connected to the frame at other locations, such as the columns, unless such attachment allows sufficient relative movement. The wall rotates about its base for lateral drift perpendicular to its plane, and moves as a relatively rigid body with the lower floor for in-plane lateral drift.

7.3.5 Durability

The current standard practice for design of masonry cavity walls assumes the cavity will be wet and incorporates a continuous waterproofing membrane on the exterior face of the back-up (Brock, 2005). Designers must balance the reliability and life expectancy of the membrane with the durability of the façade support structure behind it. The shelf angle and support structure should be located on the dry side of a reliable membrane and flashing system.

Structure on the dry side of the membrane is generally not galvanized or given protection greater than the primary building structure, which is often not protected. A usual exception is the shelf angle itself, which is protected because it is at the base of the cavity where the most water is expected to accumulate. The flashing that protects the horizontal leg has splices that may be susceptible to leakage. Water that leaks past the flashing will tend to sit on the horizontal leg and not drain away. For these reasons, the shelf angle is often galvanized.

7.4 DESIGN RESPONSIBILITIES FOR MASONRY CAVITY WALLS

As discussed in Chapter 3, it is important that the design team understands the design responsibilities for the support of masonry cavity walls. Contracts can define the design responsibilities for each project, and one approach commonly found on projects with masonry cavity walls is discussed in the following text.

Architect—The architect normally has responsibility for the following:

- The performance of the cavity wall with respect to moisture and thermal protection of the building, and to the performance of the veneer, including anchorage to the back-up system.

- The selection of cavity wall dimensions, including face of back-up to exterior face of veneer, thickness of veneer, veneer overhang, and length of horizontal leg of shelf angle.

- The location of horizontal soft joints (and thus the location of the shelf angles).

- The dimension of the horizontal soft joint and performance of sealant joint (usually in consultation with the SER to understand practical limits for controlling movement due to the steel frame and shelf angle assembly).

- The selection of the back-up wall material (block versus metal studs), and other attributes, such as maximum stud spacing and minimum gage based on fastening of exterior sheathing, interior finishes, and veneer anchors.

- The selection of the back-up wall thickness and structural performance requirements (usually in consultation with the SER).

- The back-up wall plan location with respect to column and spandrel center lines.

- The selection of the strategy for supporting the back-up and veneer (usually in consultation with the SER).

- The setting of acceptable tolerances for erected and adjusted location of the shelf angle.

Structural Engineer of Record (SER) —The SER normally has responsibility for the following:

- The design of the primary building structure, including the slab, slab edge detail, spandrel beam, roll beams, kickers, etc., to support the forces imposed by the masonry cavity wall system with due consideration of stiffness requirements.

- The design of the shelf angle and its attachment to the structure, including hangers, kickers, and other structural steel elements to support the veneer and back-up.

- The design of the connections of these elements, with details that address each type of project condition.

- Providing for field adjustment of the shelf angle and slab edge as necessary to address fabrication and erection tolerances.

- Providing the architect with structural design deformations at the shelf angle for use in designing the soft joints.

- Providing the architect with the vertical and in-plane movements that the top of the back-up wall connections must accommodate for structural deformations for inclusion in the back-up wall specifications and/or design.

- Indicating on the structural drawings pertinent assumptions and limitations about the loads from the masonry cavity walls.

- Indicating load support points on the contract drawings.

- The review and approval of shop drawings and field erection drawings for the effect of the back-up wall and attachments on the primary building structure.

Although the SER may not be under contract to design the back-up wall, the SER must understand the strategy for anchoring the back-up wall and design the primary building structure to accommodate this strategy. The SER should indicate pertinent assumptions and any limitations.

Back-up Wall Designer—For metal stud back-up walls, project documents often specify that the contractor hire a specialty structural engineer (SSE) to design the back-up. The SSE designs and specifies the back-up wall and its anchors to the primary building structure to meet performance requirements provided in the project specifications.

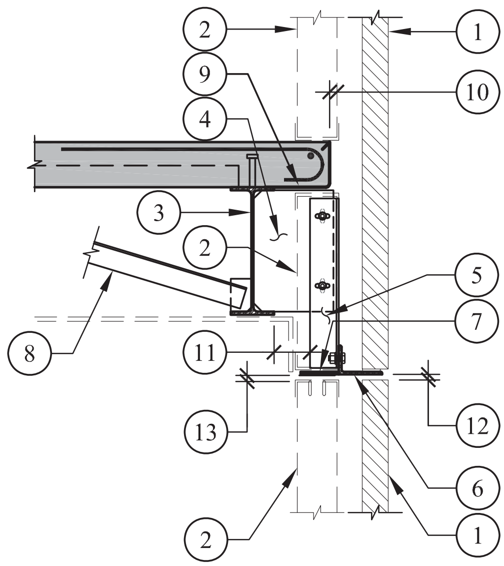
For block back-up, the practice of using an SSE for design of the block and its anchorage is less consistent. For many projects the design team will provide the back-up design including reinforcement, slab dowels, and top of wall anchorage. Whether it is shown on the structural drawings or architectural drawings depends on the contractual agreement of the design team. Top-of-wall anchorage details often call for steel plates, bent plates, or angles. If they are shown on the structural drawings and are structural steel items as defined in AISC *Code of Standard Practice*, Section 2.1, they are usually procured with the steel package. They may be installed by either the steel erector or the mason, depending on project conditions and choice of the construction manager. The actual intention of the design team for procurement and installation should be well defined in the contract documents.

7.5 DESIGN OF SHELF ANGLES

Grimm and Yura (1989) and Tide and Krogstad (1993) provide methods to design shelf angles. While both papers account for arching of masonry and flexural and torsional deformations of the shelf angle, Grimm and Yura simplified many of the calculations to ease hand calculations. However their method also contains limitations, the most notable of which are that the outstanding leg of the angle must be the long leg, and the spacing of shelf angle supports must not exceed 36 in. Tide and Krogstad provides a method that is computationally more involved, but does not have the same restrictions. Tables 7-1 through 7-5 (see end of Chapter 7) provide shelf angle mid-span tip deflections for various angle shapes, loads, and spans based on the method provided by Tide and Krogstad (1993).

7.6 HUNG SHELF ANGLE—BACK-UP SUPPORTED BY SLAB

An illustration of a hung shelf angle detail is shown in Figure 7-7. In this figure the shelf angle hangs a modest distance below the bottom flange of the spandrel beam. Kickers to the bottom flange of the spandrel, or alternatively roll beams, resist twisting of the spandrel, allowing the finished ceiling to be above the shelf angle elevation. Important attributes of this detail include:



(Note: Sheathing and Insulation Not Shown for Clarity, Typ.)

NOTES:

1. Veneer of cavity wall. Membrane, insulation, flashing, etc. not shown.
2. Block or metal stud back-up wall.
3. Spandrel beam.
4. Full-depth stiffener plates provide brackets that project from the spandrel beam to pick up the hanger.
5. Single-angle hangers. Erection bolts in horizontal long slots in the angle and vertical long slots in the bracket plates allow for field adjustment prior to field welding.
6. Shelf angle. May be shop welded if hangers have sufficient adjustment. Can have erection bolts in slotted holes with field welding after final placement for additional adjustment.
7. Continuous plate to support back-up wall above movement joint. Welded to hangers.
8. Kickers or roll beams restrain twist of spandrel.
9. Field installation of light-gage metal pour stop provides adjustment of slab edge location.
10. Nominal overhang of the back-up allows for field adjustment of the face of backup relative to the slab edge.
11. Clearance is required between the inside edge of the hanger and the outside tip of the spandrel flange.
12. Horizontal soft joint in the veneer.
13. Back-up connection to hanger assembly provides out-of-plane restraint only. Allows vertical and in-plane movement.

Fig. 7-7. Hung shelf angle with back-up wall supported on slab.

- Field application of a light-gage metal pour stop provides adjustment of the slab edge location. The slab is designed to cantilever over the spandrel and pick up the weight of the back-up. If the back-up load is too large for the slab, a steel bent plate or angle can be used. It is designed as a cantilever from the spandrel flange, or supported by the hanger stiffeners and designed to span between them.

- Full-depth stiffener plates provide brackets that project from the spandrel beam to pick up the hanger. Typical spacing of the bracket plates ranges from 3 to 5 ft, depending on project conditions.

- The hangers are single steel angles, though double angles, WT sections, channels, and HSS are other common hanger shapes. Erection bolts in long horizontal slots in the hanger and long vertical slots in the bracket plates allow for field adjustment prior to welding. A single hanger on one side of the bracket is preferred as this allows field welding on both the near and far sides of the hanger. Designers can use slip-critical bolts for final connections, but plate washers will be required on both sides due to the slotted holes.

- The hangers are designed for flexure as well as tension. The flexure comes from both the eccentric veneer load and the horizontal forces from the veneer and the back-up. In particular, the horizontal out-of-plane force from the top of the back-up wall of the story below is applied at the bottom of the hanger. Alternatively, kickers can be used to resist this force.

- Clearance is required between the inside edge of the hanger and the outside tips of the spandrel flanges, and should account for the field adjustment.

- The shelf angle can be shop welded or bolted to the hanger because the adjustability is provided between the hanger and the bracket plate. Designers can choose to provide additional field adjustment with erection bolts and slotted holes. It is usually not sufficient to try to provide the only adjustment at the shelf angle to hanger connection because the "in-and-out" adjustment has to be made with shimming, and this can be cumbersome and problematic.

- Planning for a nominal overhang of the back-up but designing it to be either zero or twice as large as theoretically required allows for field adjustment of the face of the back-up relative to the slab edge. For example, planning on 1 in. but designing for 2 in. of overhang will provide for a 1 in. adjustment in or out. This adjustment of the back-up location can lessen the amount of slab edge adjustment needed.

- The back-up wall is anchored to the top of the slab. If the back-up is a block wall, the primary building structure usually includes dowels from the slab. If the back-up is metal stud framing, the anchors to the slab are usually post-installed concrete anchors and not part of the primary building structure.

- A short height of back-up wall is supported from the hanger assembly and spans to the underside of the slab or deck. Clearance should be provided between the outside tips of the spandrel flanges and the backside of the back-up for tolerance and fireproofing.

- A continuous plate, angle, or other element shop attached to the hangers can be used to provide a means to support the short height of back-up, and a place to anchor the top of the back-up wall below the window head.

- The connection of the back-up wall to the underside of the hangers must resist out-of-plane forces from the wall but allow sufficient relative movement between the wall and the hanger assembly. The relative movements include both the vertical movement due to differential deflection of the floors and the in-plane movement due to story drift of the frame.

An example structural detail of a hung shelf angle is provided in Figure 7-8.

The locations of the hangers should be planned by the design team. Such a typical detail provides the maximum spacing of the hangers covering most conditions for most projects. However, the detailer should still be given direction by the designers on special conditions, such as near the ends of the spandrels at columns, at exterior corners, and at jogs or other articulations in the perimeter building line. Figure 7-9 provides examples of such conditions.

An example of a hung shelf angle detail with the shelf angle hung a large distance below the spandrel is shown in Figure 7-10. In the example, project conditions allow kickers to extend below the spandrel beam and close to the bottom of the hangers. Important attributes of this detail are:

- For adjustment of the hanger assembly, adjustment must be provided in the connection of the kicker to the hanger as well as in the connection of the hanger to the spandrel.

- Longer hangers will allow additional rotation of the shelf angle if their size is not increased relative to shorter hangers. If the kicker-to-hanger connection is designed as a pin, only the flexural stiffness of the hanger resists rotation. If the connection is designed as rigid, the designer can account for the flexural stiffness of the kicker as well, as illustrated in Figure 7-11.

- The twist of the spandrel beam due to the eccentricity of the hangers should be considered. To be concentric with the spandrel, the hanger can be pulled in or the spandrel can be pushed out. If the hanger is moved in, a bracket with adjustability at the shelf angle is needed. This will increase the rotation that the hanger and kicker need to resist. If the spandrel is moved outward, it will interrupt the back-up wall. The designer must understand how the sheathing, waterproofing, and veneer anchors will be laterally supported over the depth of the spandrel, as well as the impact this may have on the fireproofing of the spandrel.

- The horizontal force on the kicker may be large because the tributary area may be large. The interior beam and slab must resist the uplift and the horizontal component must be transferred into the slab and deck.

For long hangers, it may be more economical to provide fewer, heavier hangers using HSS to span between them and support the shelf angle. Perhaps the hangers can be located at the quarter- or third-points of the spandrel beam. The HSS/shelf angle assembly is supported by the hangers, and by brackets from the columns. Figure 7-12 illustrates this concept.

An alternative to using kickers for a hung shelf angle detail with long hangers is shown in Figure 7-13. This example eliminates the kickers and adds a horizontal girt at the base of the hanger. Important attributes of this detail are:

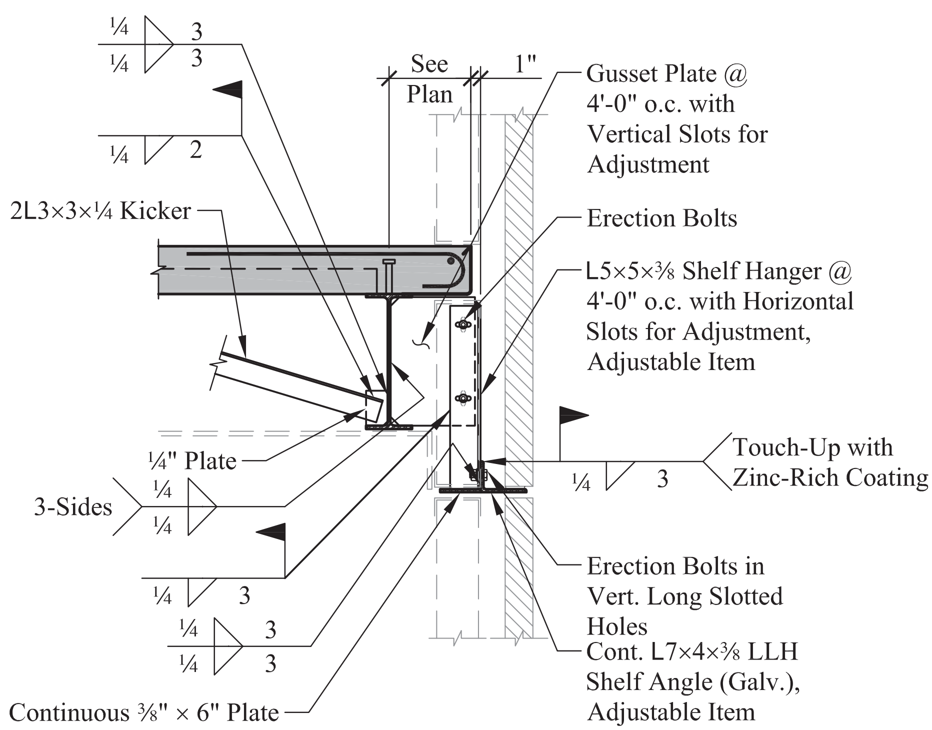
- The gravity loads are taken to the spandrel by the hangers. The girt spans between columns to resist out-of-plane wall forces from the wall above and below the girt.

- The girt also resists a horizontal force that results from the vertical loads being eccentric from the hanger. The flexural stiffness of the hanger resists the rotation of the shelf angle. Designers should also consider the lateral displacement of the girt. Limiting the lateral deflection of the girt to L/600 to L/720 is sufficient for most masonry veneers.

- There is still twist of the spandrel beam due to the hanger being eccentric.

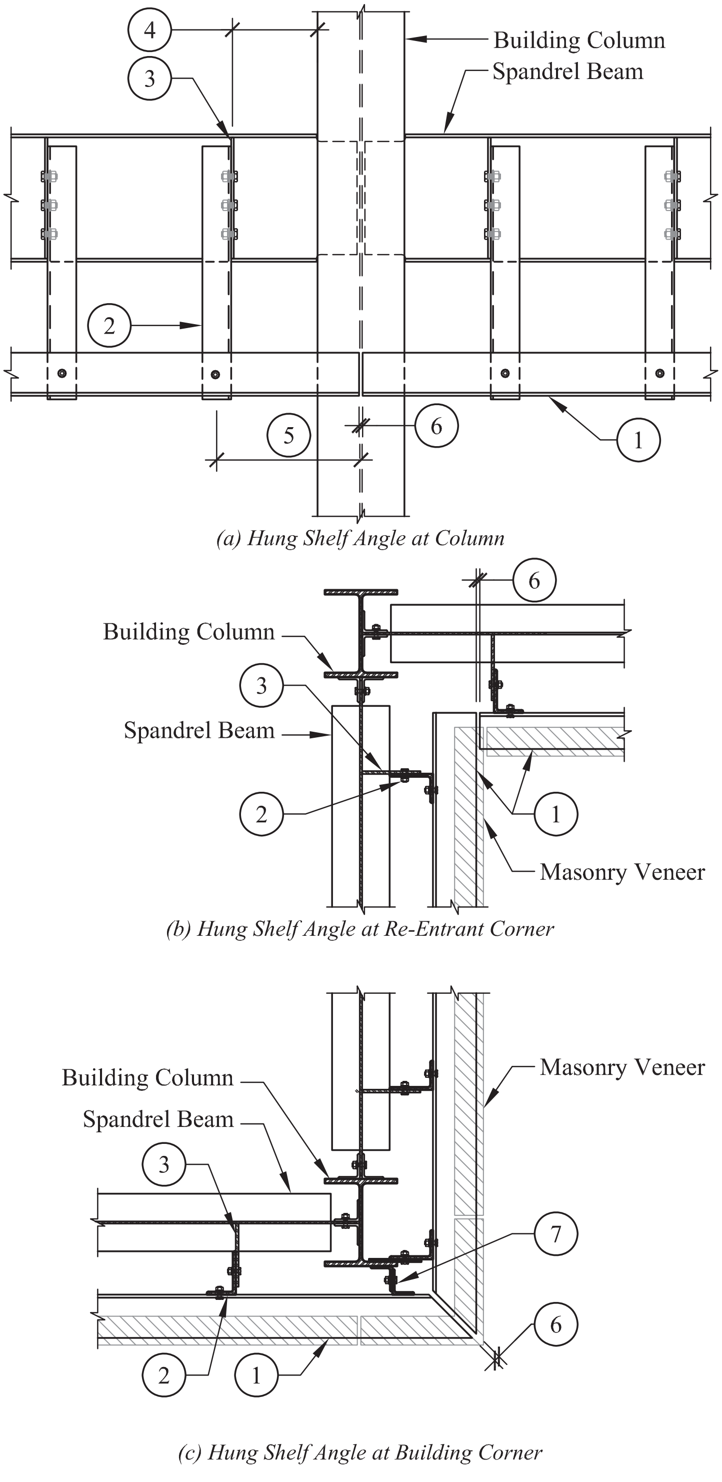
- The finished ceiling can be high relative to the shelf angle since there are no kickers. However, the architectural finish must account for the girt either by making a soffit around it or thickening the wall.

Example 7.3 illustrates the design concepts for a hung shelf angle detail.



(Note: Sheathing and Insulation Not Shown for Clarity, Typ.)

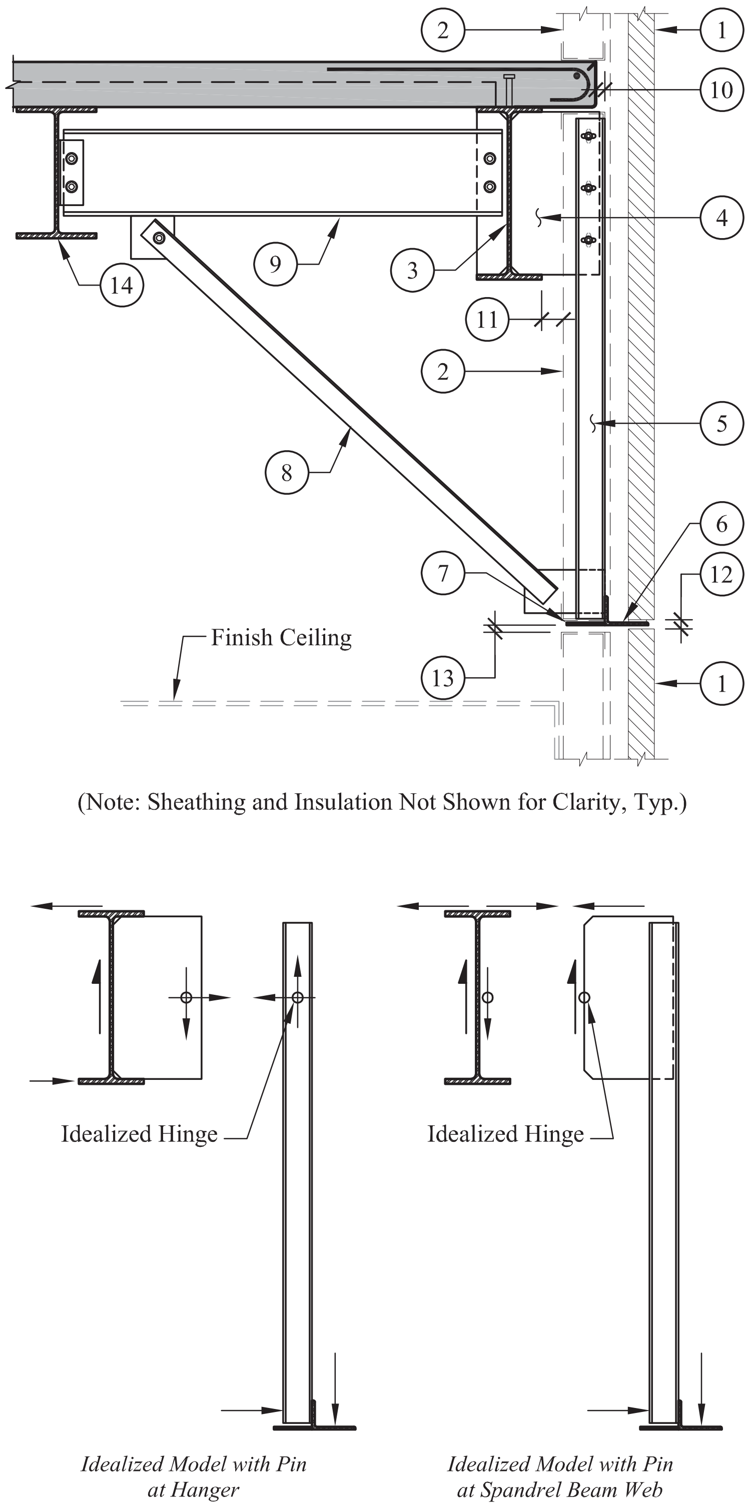
Fig. 7-8. Example structural detail—hung shelf angle.



NOTES:

1. Shelf angle.
2. Hangers.
3. Brackets from spandrel to hangers.
4. Allow clearance from last bracket hanger to column for column connection; 12 to 18 in. is usually sufficient.
5. Design shelf angle for cantilever past last hanger on spandrel; 18 to 24 in. is not unusual.
6. Gap between shelf angles is ½ in. +/- ½ in. Gaps in angles and vertical control joints need not align.
7. Field install adjustable brackets from column to support ends of shelf angles if cantilever from last bracket is too long.

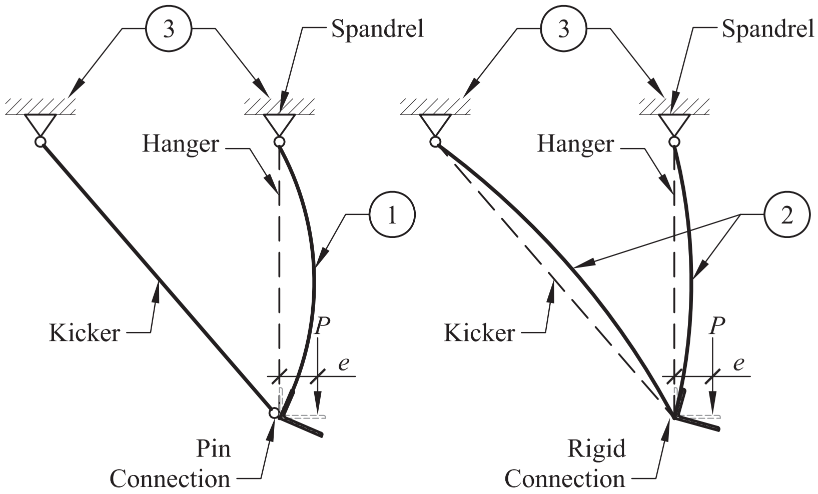
Fig. 7-9. Hung shelf angle details.



NOTES:

1. Veneer of cavity wall. Membrane, insulation, flashing, etc. not shown.
2. Block or metal stud back-up wall.
3. Spandrel beam.
4. Full-depth fitted stiffener plates provide brackets that project from the spandrel beam to pick up the hanger.
5. Hangers. Erection bolts in horizontal long slots in the angle and vertical long slots in the bracket plates allow for field adjustment prior to field welding.
6. Shelf angle. May be shop welded if hangers have sufficient adjustment. Can have erection bolts in slotted holes with field welding after final placement for additional adjustment.
7. Continuous plate to support back-up wall above movement joint. Welded to hangers.
8. Kickers. Field weld to connection plates after adjustment of hangers.
9. Roll beam to restrain twist on spandrel due to eccentricity of hanger. Can also help transmit force from kicker into slab.
10. Nominal overhang of the back-up allows for field adjustment of the face of back-up relative to the slab edge.
11. Clearance is required between the inside edge of the hanger and the outside tip of the spandrel flange.
12. Horizontal soft joint in the veneer.
13. Back-up connection to hanger assembly provides out-of-plane restraint only. Allows vertical and in-plane movement.
14. Interior beam resists vertical force from kicker. Consider bottom flange may be in compression and is unbraced if there is net uplift.

Fig. 7-10. Hung shelf angle with long hangers and kickers.



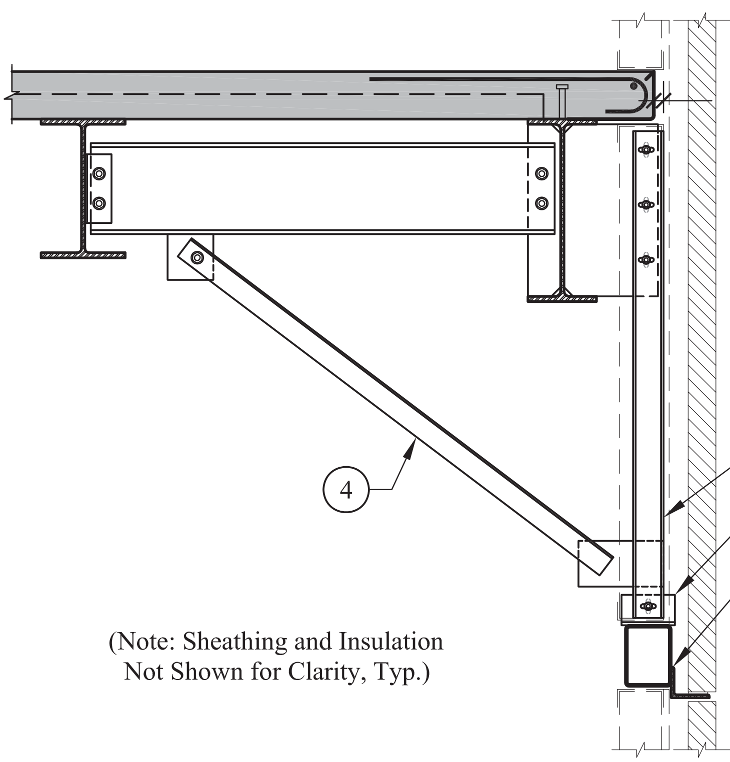
NOTES:

1. If connection of the kicker to the hanger is a pin, only the hanger resists rotation of the shelf angle. Check stiffness and strength of hanger.

2. If connection of the kicker to the hanger is rigid, the hanger and kicker resist rotation of the shelf angle. Check stiffness and strength of both.

3. There can be differential deflection between supports, which will impact shelf angle deflection.

Fig. 7-11. Rotation in hung shelf angle due to flexure in hangers and kickers.



NOTES:

1. Use fewer, heavier hangers, perhaps at ¼ or ⅓ span of the spandrel beam. Double angles or channels may be appropriate.

2. HSS + shelf angle assembly spans between hangers. HSS takes torsion from eccentric shelf angle. Support HSS at columns to avoid heavy hangers adjacent to columns.

3. Design connection between HSS and hangers for vertical and horizontal field adjustment. Consider erection bolts in slotted holes and field welding.

4. Kicker.

Fig. 7-12. Alternative strategy for hung shelf angle with long hanger and kickers.

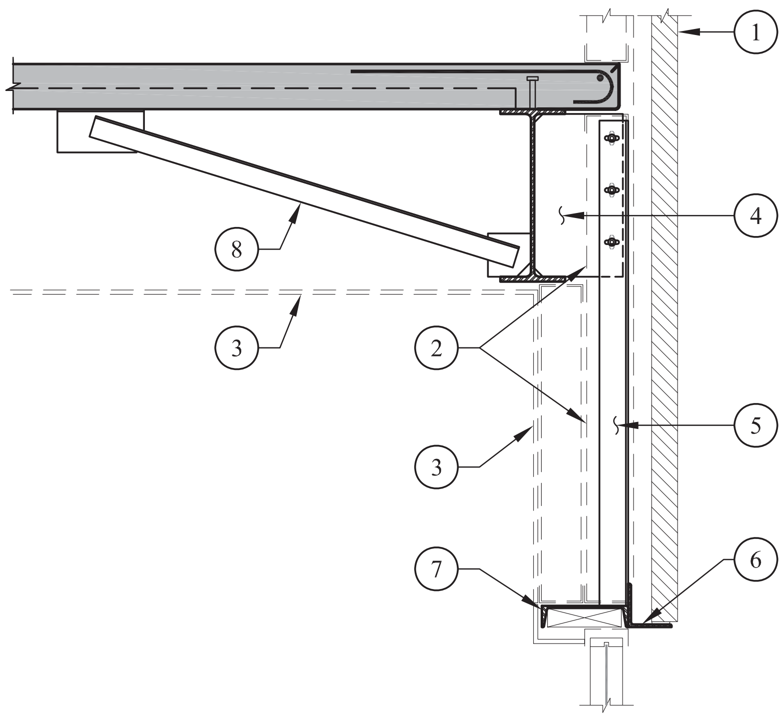
7.7 HUNG SHELF ANGLE—BACK-UP RUNS BY SLAB EDGE

For cavity walls made with block back-up, the back-up is usually supported on the slab as discussed in the previous section. When the back-up wall is made of metal studs, designers have the option of running the studs past the edge of the slab. This is ideal for parapets and strip windows because the studs can cantilever by the edge of the slab up to the window or parapet.

Figure 7-14 shows an example detail of a hung shelf angle with the metal stud back-up running by the edge of the slab. Important attributes of this detail are:

- The shelf angle is hung from steel bracket plates extending from the spandrel beam with single-angle hangers. The hanger assembly and its connections are similar to those discussed in Section 7.5. The benefit of supporting the shelf angle with structural steel (as opposed to fastening it directly to the metal studs) is that the shelf angle is then erected and adjusted by the steel erector.

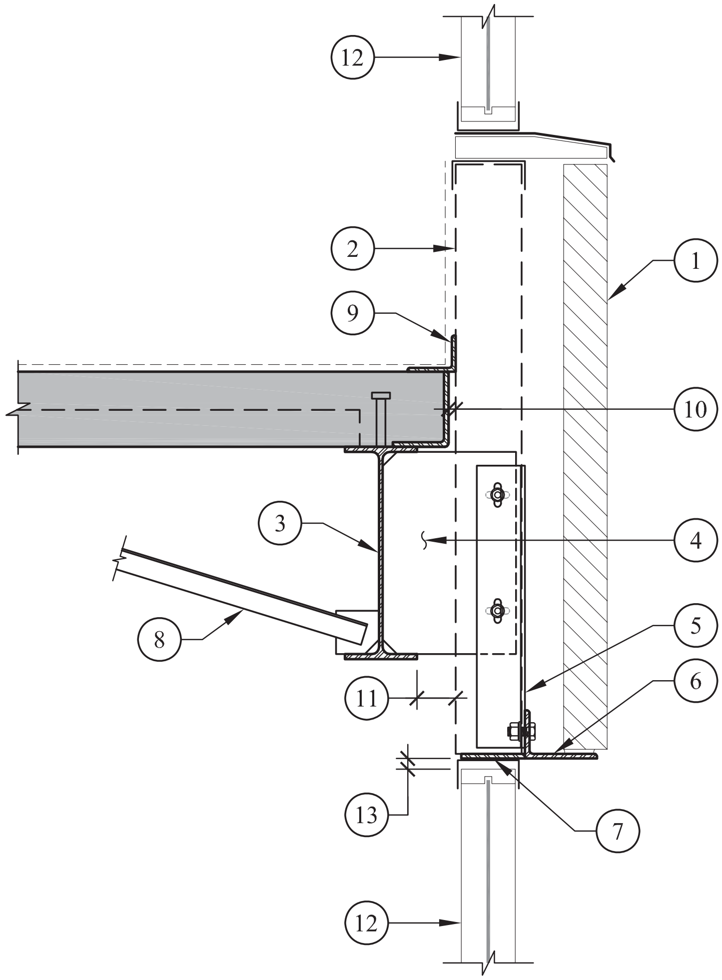
- The back-up wall weight is supported on the hanger assembly. Therefore, the connection to the slab edge need only be for the out-of-plane forces.



NOTES:

1. Veneer of cavity wall. Membrane, insulation, flashing, etc. not shown.
2. Block or metal stud back-up wall.
3. Finish may prevent the use of kickers down to shelf angle.
4. Full-depth stiffener plates provide brackets that project from the spandrel beam to pick up the hanger.
5. Hangers. Erection bolts in long horizontal long slots in the angle and vertical long slots in the bracket plates allow for field adjustment prior to field welding.
6. Shelf angle. May be shop welded if hangers have sufficient adjustment. Can have erection bolts in slotted holes with field welding after final placement for additional adjustment.
7. Continuous girt spans between columns for out-of-plane wall loads and the horizontal force that results from eccentric load on hanger. Also supports the back-up wall above movement joint. Welded to hangers for vertical support.
8. Kicker to resolve eccentric forces on spandrel beam.

Fig. 7-13. Hung shelf angle with long hangers without kickers.



(Note: Sheathing and Insulation Not Shown for Clarity, Typ.)

NOTES:

1. Veneer of cavity wall. Membrane, insulation, flashing, etc. not shown.
2. Metal stud back-up wall runs past edge of slab.
3. Spandrel beam.
4. Full depth stiffener plates provide brackets that project from the spandrel beam to pick up the hanger.
5. Hangers. Erection bolts in horizontal long slots in the angle and vertical long slots in the bracket plates allow for field adjustment prior to field welding.
6. Shelf angle. May be shop welded if hangers have sufficient adjustment. Can have erection bolts in slotted holes with field welding after final placement for additional adjustment.
7. Continuous plate to support back-up wall above movement joint. Welded to hangers.
8. Kickers or roll beams to restrain twist on spandrel due to eccentricity of hanger.
9. Metal studs have lateral anchorage by means of a continuous clip to top of slab, or individual clips for each stud to edge of slab.
10. Nominal gap by design between back-up and slab edge allows for field adjustment of the face of back-up relative to the slab edge.
11. Clearance is required between the inside edge of the hanger and the outside tip of the spandrel flange.
12. Windows can be strip windows in this detail as the studs sit on the hanger assembly and cantilever up past the edge of slab.
13. Window head connection to hanger assembly provides out-of-plane restraint only. Allows vertical and in-plane movement.

Fig. 7-14. Hung shelf angle with metal studs running past slab edge.

- The metal studs can be connected to the slab edge by anchoring to the top of slab (shown) or to the front edge of the slab.

If the metal studs are connected to the top of the slab, the designer may choose to form the edge of the slab with a light-gage metal pour stop. A continuous cold-formed metal angle is attached to the back of the studs and anchorage to the top of the slab is provided with post-installed anchors. Alternatively, steel embed plates can be provided in the slab, though this will likely require a steel angle or bent plate to form the slab edge so as to have a means to hold the embeds in place during concrete operations.

If the metal studs are connected to the front edge of the slab, a steel angle or bent plate must be designed. A connection clip should be provided for each individual stud. The clips are usually field welded to the steel edge as it is difficult to drill and place concrete anchors through the steel edge member.

A gap should be provided between the back of the stud and the slab edge. This allows for field adjustment of the location of the back-up wall relative to the slab edge. The amount of adjustment necessary depends on the degree to which the field adjustment of the slab edge addresses the steel frame tolerances.

The shelf angle can be supported directly from the metal studs. This has the advantage of eliminating the steel bracket plates and hanger assemblies. It also facilitates erecting the back-up wall in pre-assembled panels with the shelf angle shop applied to the studs. However, a disadvantage is that all the field adjustment must now occur in the connection of the panel to the slab. The metal stud-to-slab edge connection must also carry the weight of the back-up panel and subsequent veneer. It is also difficult to weld unless the cold-formed studs are made of fairly heavy gage material, such as 16 gage or heavier.

7.8 SHELF ANGLE SUPPORTED AT SLAB EDGE

Examples of shelf angles supported at a slab edge are shown in Figure 7-15. Supporting the shelf angle at the slab edge is usually best when the project already has a robust slab for other reasons. The elevation of the horizontal leg of the angle relative to the slab elevation is crucial. An inch or two up or down relative to the slab is the difference between a clean and sometimes awkward connection detail.

Important attributes of details for supporting the shelf angle at the slab edge are:

- The slab should be designed with adequate shear and flexural strength to overhang the spandrel and support the wall. Otherwise, a very heavy bent plate or angle is required and it may still need to be stiffened. With the slab properly designed to support the wall, the spandrel does not have the tendency to twist. If the slab is not adequate and a plate or bracket cantilevers from the spandrel, kickers or rolls beams may be necessary to resist the torsion in the spandrel.

- A steel angle or bent plate should be provided as a pour stop and a means to connect the shelf angle to the slab. Ample in-and-out field adjustment should be provided for this bent plate.

- Headed studs or deformed bar anchors with threaded couplers welded to the vertical leg of the bent plate should be used to transfer the veneer load to the slab.

- The shelf angle should be field welded to the bent plate pour stop. Erection bolts and long slotted holes can be used in the vertical leg of the shelf angle for vertical adjustment in the field. Adequate space must be provided for field weld, accounting for the vertical adjustment of the shelf angle.

- Clearance should be provided between the back-up wall and the outside tips of the spandrel beam flanges to allow the back-up wall to be connected to the underside of the bent plate at the slab overhang. This connection must transfer out-of-plane forces from the wall to the slab but also allow both vertical movement between the slab and the lower back-up wall and in-plane movement of the wall relative to the slab for story drift of the frame.

Example 7.5 illustrates the design concepts for a shelf angle supported at the slab edge.

7.9 POTENTIAL PROBLEMS WITH SUPPORT AND ANCHORAGE OF MASONRY CAVITY WALLS

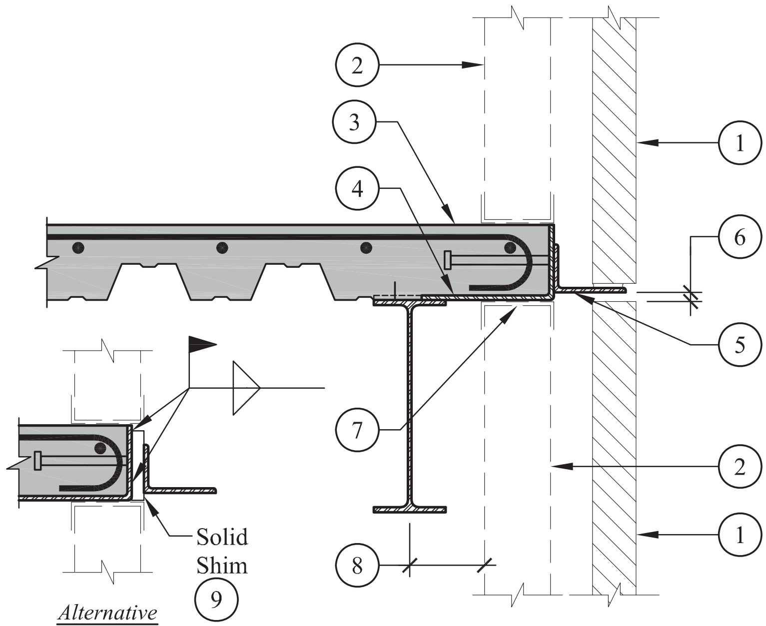
The following is a list of potential problems that designers should be aware of and avoid when designing the support and anchorage systems for masonry cavity walls. There is no particular meaning implied by the order of the list.

- Inadequate provisions for the shelf angle adjustment can cause the masonry to have too little bearing on the shelf angle.

- Inadequate provisions for the shelf angle adjustment can cause the masonry cavity to be too wide for the specified masonry ties.

- Improper flashing design may not accommodate the projection of bolts or fasteners into the cavity at the shelf angle.

- Inadequate soft-joint size may cause over compression of the sealant, causing it to bulge out.



NOTES:

1. Veneer of cavity wall. Membrane, insulation, flashing, etc. not shown.

2. Metal stud or block back-up wall.

3. Design the slab with adequate shear and flexural strength to overhang the spandrel and support the wall.

4. Provide a steel angle or bent plate as a pour stop, and as a means to connect the shelf angle to the slab edge with headed studs or deformed bar anchors. Design ample in-out field adjustment of this bent plate. For additional adjustment detail for single, solid shim between angle and slab edge.

5. Field weld the shelf angle to the bent plate. If alternative shim detail is used, weld shim to bent plate and angle to shim.

6. Soft joint in veneer.

7. Anchorage of back-up to slab. This connection needs to transfer out-of-plane forces from the wall to the slab but allow vertical movement between the slab and the lower back-up wall, and in-plane movement of the wall relative to the slab for story drift of the frame.

8. Provide clearance between the back-up wall and the outside tip of the spandrel beam flange to allow the back-up wall to be connected to the underside of the bent plate at the slab overhang.

9. Solid shims of varying thicknesses provide additional field in-plane/out-of-plane adjustment.

Fig. 7-15. Relieving angle supported on slab edge.

- Inadequate clearance between the bottom of the shelf angle and the top of the masonry veneer may result in bearing of the shelf angle on the masonry due to spandrel deflections.
- Support details for the shelf angles at corners and atypical conditions are not clearly documented in the design.

Example 7.1—Determination of Deflections for Structures Supporting Brick Veneers

For the clay brick veneer on a building supported at each floor level by a shelf angle, determine the amount by which the brick will expand and the amount by which the spandrel beam can deflect before compressing the joint filler material to its maximum allowable compressibility.

Given:

The brick height between the shelf angles, h = 14 ft. The vertical as-built construction tolerance of the building, Tol = ⅛ in. The shelf angle deflection under the weight of the brick with respect to the floor below, δ_a = 1⁄16 in.

The brick veneer is installed at an ambient temperature of 40 °F. The exterior wall of the building may experience a change in temperature, $\Delta_T = 100$ °F. The brick coefficient of thermal expansion, $\alpha = 4 \times 10^{-6}$ in./in./°F, and the brick coefficient of moisture expansion, $k_e = 3 \times 10^{-4}$ in./in.

The largest design gap width before sealant installation, J' = ⅞ in. The joint filler material is a high-performance sealant and the compressibility of the sealant material, M = 50%.

Solution:

The volume change of the brick between the shelf angles is,

$$\begin{aligned}\delta_{vb} &= k_e h + \alpha \Delta_T h \\ &= (3 \times 10^{-4} \text{ in./in.})(14 \text{ ft})(12 \text{ in./ft}) \\ &\quad + (4 \times 10^{-6} \text{ in./in./°F})(100 \text{ °F})(14 \text{ ft})(12 \text{ in./ft}) \\ &= 0.118 \text{ in.}\end{aligned}$$

The amount of movement that the joint can accommodate is,

$$\begin{aligned}\delta_{jm} &= M\left(J' - Tol\right) \\ &= 0.50\left(\tfrac{7}{8} \text{ in.} - \tfrac{1}{8} \text{ in.}\right) \\ &= 0.375 \text{ in.}\end{aligned}$$

Thus, the permissible structural deflection, including the deflection due to the loads applied prior to installation of the sealant joint is,

$$\begin{aligned}\delta_s &= \delta_{jm} - \delta_{vb} \\ &= 0.375 \text{ in.} - 0.118 \text{ in.} \\ &= 0.257 \text{ in.}\end{aligned}$$

The permissible deflection for the spandrel beam includes both vertical deflections and torsional rotations as described in Chapter 5. Note that this deflection, δ_s, is less than L/600 for beams spanning greater than 12 ft 10 in.

The total structural deflection is $\delta_{sil} + M\delta_{ps}$, where

$$\delta_{ps} = \delta'_{sb} + \delta_a$$

The deflection of the spandrel beam, δ'_{sb}, due to the brick load and δ_a is the total deflection of the shelf angle due to the brick load. Substituting this quantity back into the equation for the total structural deflection, and limiting the total to not exceed δ_s,

$$\delta_{sil} + M\delta'_{sb} + M\delta_a \le \delta_s$$

The deflection of the spandrel beam is proportional to the load on it. Knowing that the uniformly distributed load due to the brick on the spandrel is w_{sb} and the superimposed load is w_{sil}, the deflection of the spandrel beam due to the uniformly distributed load due to the brick is,

$$\delta'_{sb} = \delta_{sil} \frac{w_{sb}}{w_{sil}}$$

This ratio assumes that the interior beams are parallel to the spandrel beam and do not frame into the spandrel beam. The assumption may still be a reasonable approximation when floor beams frame to the spandrel beam. Substituting this relationship back into the equation for δ_s above gives,

$$\delta_{sil} + M\left(\delta_{sil} \frac{w_{sb}}{w_{sil}}\right) + M\delta_a \le \delta_s$$

Rearranging this for δ_{sil}, the amount the spandrel beam can deflect is,

$$\begin{aligned}\delta_{sil} &\le \frac{\delta_s - M\delta_a}{1 + M\left(\dfrac{w_{sb}}{w_{sil}}\right)} \\ &\le \frac{0.257 \text{ in.} - 0.50\left(\tfrac{1}{16} \text{ in.}\right)}{1 + 0.50\left(1\right)} \\ &\le 0.151 \text{ in.}\end{aligned}$$

Comments:

This example shows how to determine the permissible structural deflection when considering the size of soft (control) joints for brick veneers. Although the design of this joint generally is not in the scope of work for the SER, a general knowledge of proper joint design is often useful for the SER.

The design gap before sealant installation may be expressed by the following equation:

$$J' = \frac{\alpha\Delta_T h + k_e h + \delta_{sil}}{M} + Tol + \delta_{ps}$$

where

δ_{ps} = the relative deflection of the structure with respect to the brick below the shelf angle that occurs after the shelf angle is set but before sealant installation

δ_{sil} = the deflection due to superimposed loads applied to the spandrel beam supporting the shelf angle after the sealant has been installed in the joint

This expression can be rearranged to determine the total deflection that can be tolerated:

$$\delta_{sil} + M\delta_{ps} = M\left(J' - Tol\right) - \left(\alpha\Delta_T h + k_e h\right)$$

Note that the shelf angle deflection includes deflection of the horizontal leg of the angle as well as the deflection of the relieving angle between attachments to the building structure.

For coefficients of thermal and moisture expansion, refer to American Concrete Institute ACI 530-05, *Building Code Requirements for Masonry Structures*, for coefficients for masonry. The Brick Industry Association also provides excellent guidance (Brick Industry Association, 2000).

This calculation requires the designer to make assumptions about the brick and superimposed loads on the beam, which may be difficult during preliminary design. In practice, it is conservative and often easier to use the total of the brick deflection and the superimposed load deflection such that:

$$\delta_s = \delta_{ps} + \delta_{sil}$$

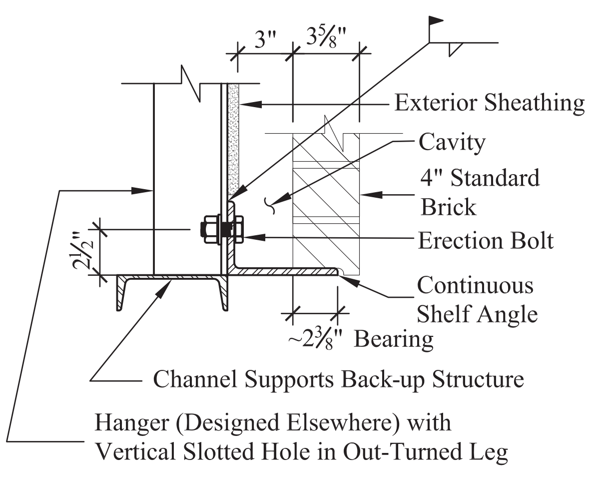


Fig. 7-16. Section of shelf angle supporting brick veneer.

Note that at the first elevated floor level where the brick below the shelf angle is supported on a foundation wall, δ_{ps} may be significant. At upper floor levels, however, where the spandrel beam on the floor below the shelf angle may deflect approximately as much as the spandrel beam supporting the shelf angle in question, δ_{ps} may be small. Thus, one can often conservatively select a beam for which the total deflection associated with cladding loads, superimposed dead loads, and live loads is less than δ_s.

Example 7.2—Selection of Shelf Angles to Support Brick Veneer Cladding

For the clay brick veneer on a building supported at each floor level by a shelf angle as illustrated in Figure 7-16, determine a shelf angle size that meets the deflection criteria.

Given:

The brick height between the shelf angles, h = 14 ft. The shelf angle is supported by hangers at 3 ft on center along its length. The cavity width between the exterior face of the vertical leg of the shelf angle and the interior face of the brick, w_c = 3 in. The width of the brick, w_b = 3⅝ in. The sheathing thickness, assuming the angle is thinner than the sheathing and the back of the vertical leg of the angle aligns with the back of the sheathing, t_{sh} = ⅝ in.

For architectural reasons and to accommodate vertical deflection of the supporting structure, the tip deflection of the angle is limited to 1⁄16 in. Assume that lateral wind forces are resisted entirely by the back-up structure.

Solution:

The required horizontal leg length of the shelf angle such that approximately two-thirds of the brick width sits on the shelf angle is,

$$\begin{aligned} l_{ah} &= \tfrac{2}{3} w_b + w_c + t_{sh} \\ &= \tfrac{2}{3}(3\tfrac{5}{8}\text{ in.}) + 3\text{ in.} + \tfrac{5}{8}\text{ in.} \\ &= 6.04\text{ in.} \end{aligned}$$

Use an angle with a 6-in.-long horizontal leg.

Enter Table 7-3 (14-ft height of brick) with the 6-in. horizontal leg length, the 1⁄16-in. maximum tip deflection and 3-ft support spacing. An L6×4×⅜ LLH has a maximum tip deflection of 0.0625 in., which satisfies the 1⁄16-in. limit.

Use L6×4×⅜ LLH.

Comments:

The design methodology used in this example is based upon Tide and Krogstad (1993), as are the tables in Chapter 7.

Example 7.3—Shelf Angle for Brick Veneer Supported by Medium Hanger System on Floor Spandrel Beam

For the clay brick veneer on a building supported at each floor level by a shelf angle as illustrated in Figure 7-17, determine the vertical and horizontal deflections at the tip of the shelf angle. Also, estimate the additional vertical and horizontal deflections at the tip of the shelf angle due to rotation of the spandrel beams when roll beams at 9 ft on center are used in place of the kickers resisting torsion of the spandrel beam.

Given:
Use the same loads and the shelf angle designed in Example 7.2.

The shelf angle weight, w_{sa} = 12.3 lbs/ft and the horizontal leg, l_{ah} = 6 in. The floor-to-floor height between shelf angles, h = 14 ft. The shelf angle is supported at s = 3 ft on center by single-angle hangers. As determined previously from Table 7-3, the maximum vertical displacement of the shelf angle, Δ_a = 0.0625 in. The tolerance, l_{tol} = 2 in. (This value is selected based on the plumbness tolerances given in the AISC *Code of Standard Practice*. In this case, the building is of a height for which the maximum out-of-plumbness is the 2-in. tolerance away from the building line, and this value is used for the design to keep the spandrel detail consistent throughout the height of the building.) The hangers are attached to fitted bracket plates in the spandrel beam web. Kickers brace the bottom flange of the spandrel beam against twist at each hanger.

The L4×4×⅜ hanger angle has the following properties:

$$I_x = I_y = 4.32 \text{ in.}^4$$
$$\bar{x} = \bar{y} = 1.13 \text{ in.}$$

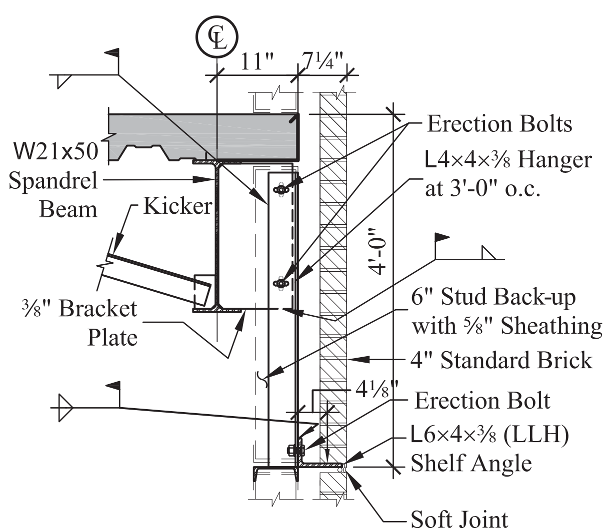


Fig. 7-17. Section of spandrel beam with hanger system.

The W21×50 spandrel beam spans 27 ft and has the following properties:

d = 20.8 in.
b_f = 6.53 in.
t_f = 0.535 in.

Under superimposed dead load and live load effects, the spandrel has deflection, Δ_s = 0.190 in.

The distance from the spandrel beam centerline to the edge of the slab, l_{eos} = 11 in. The distance between the top of the slab and the bottom of the angle, h_a = 4 ft. The slab thickness, t_s = 6¼ in.

The depth of the stud wall assembly, d_s = 6 in. The weight of the stud backup wall system, w_s = 10 psf. The wind suction on the backup, p_w = 25 psf.

The weight of the brick veneer, w_b = 40 psf. The brick load is applied to the relieving angle ½ in. outboard of the interior face of the brick. Thus, the distance from the back of the vertical leg of the shelf angle to the point of the brick load, e_b = 4⅛ in.

Solution:
The weight of brick per hanger is,

$$\begin{aligned} P_b &= w_b hs \\ &= 0.040 \text{ kip/ft}^2 (14 \text{ ft})(3 \text{ ft}) \\ &= 1.68 \text{ kips} \end{aligned}$$

The height of the stud wall supported on the hanger, neglecting the axial elongation of the hanger, is,

$$\begin{aligned} h_{st} &= h_a - t_s \\ &= 4 \text{ ft} - 6\tfrac{1}{4} \text{ in.}(1 \text{ ft}/12 \text{ in.}) \\ &= 3.48 \text{ ft} \end{aligned}$$

The weight of the stud wall supported on each hanger is,

$$\begin{aligned} P_s &= w_s h_{st} s \\ &= 0.010 \text{ kip/ft}^2 (3.48 \text{ ft})(3 \text{ ft}) \\ &= 0.104 \text{ kip} \end{aligned}$$

The lateral wind load applied at each hanger is,

$$\begin{aligned} P_w &= p_w \left(\frac{h - h_a}{2} + \frac{h_{st}}{2} \right) s \\ &= 0.025 \text{ kip/ft}^2 \left(\frac{14 \text{ ft} - 4 \text{ ft}}{2} + \frac{3.48 \text{ ft}}{2} \right)(3 \text{ ft}) \\ &= 0.506 \text{ kip} \end{aligned}$$

Both the brick and stud are eccentric to the centroid of the hanger as shown in Figure 7-18. Thus, the brick veneer and stud impose moments at the end of the hanger. However, the brick veneer and stud moments act in opposite directions.

The cantilever length of the hanger is,

$$\begin{aligned} L_c &= h_a - (d + t_s) + t_f \\ &= 4 \text{ ft}(12 \text{ in./ft}) - (20.8 \text{ in.} + 6\tfrac{1}{4} \text{ in.}) + 0.535 \text{ in.} \\ &= 21.5 \text{ in.} \end{aligned}$$

Assuming that the hanger is fixed for flexure at the bottom of the bracket plate, the moment at the bottom of the hanger due to the eccentricity of the brick is,

$$\begin{aligned} M_b &= P_b(e_b + \bar{x}) \\ &= 1.68 \text{ kips}(4\tfrac{1}{8} \text{ in.} + 1.13 \text{ in.}) \\ &= 8.83 \text{ kip-in.} \end{aligned}$$

The moment at the bottom of hanger due to the eccentricity of the stud wall is,

$$\begin{aligned} M_s &= P_s\left(\frac{d_s}{2} - \bar{x}\right) \\ &= 0.104 \text{ kip}\left(\frac{6 \text{ in.}}{2} - 1.13 \text{ in.}\right) \\ &= 0.194 \text{ kip-in.} \end{aligned}$$

The net moment at the bottom of hanger is,

$$\begin{aligned} M_{net} &= M_b - M_s \\ &= 8.83 \text{ kip-in.} - 0.194 \text{ kip-in.} \\ &= 8.64 \text{ kip-in.} \end{aligned}$$

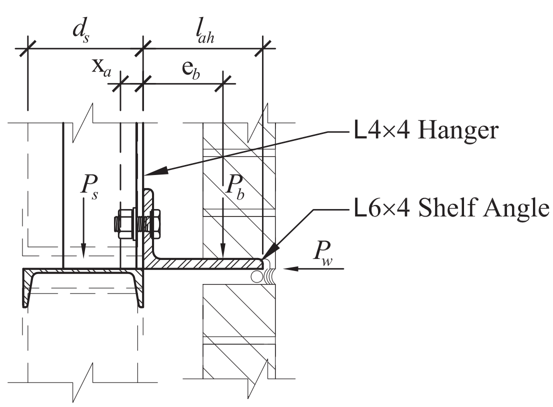


Fig. 7-18. Loads on hanger.

The rotation due to moment at the bottom of the hanger is,

$$\begin{aligned} \theta_M &= \frac{M_{net} L_c}{EI_x} \\ &= \frac{8.64 \text{ kip-in.}(21.5 \text{ in.})}{29{,}000 \text{ ksi } (4.32 \text{ in.}^4)} \\ &= 0.00148 \text{ rad. (or } 0.0850°) \end{aligned}$$

The rotation due to the wind load at the bottom of the hanger is,

$$\begin{aligned} \theta_w &= \frac{P_w L_c^2}{2EI_x} \\ &= \frac{0.506 \text{ kip}(21.5 \text{ in.})^2}{2(29{,}000 \text{ ksi})(4.32 \text{ in.}^4)} \\ &= 0.000934 \text{ rad.} (\text{or } 0.0534°) \end{aligned}$$

The horizontal distance between the centroid of the hanger and the tip of the angle leg is,

$$\begin{aligned} x_i &= \bar{x} + l_{ah} \\ &= 1.13 \text{ in.} + 6 \text{ in.} \\ &= 7.13 \text{ in.} \end{aligned}$$

The vertical displacement of the shelf angle due to rotation at the bottom of the hanger is,

$$\begin{aligned} \Delta_{v\theta} &= x_i \sin(\theta_w + \theta_M) \\ &= (7.13 \text{ in.})\sin(0.0534° + 0.0850°) \\ &= 0.0172 \text{ in.} \end{aligned}$$

The total vertical deflection of the shelf angle is,

$$\begin{aligned} \Delta &= \Delta_s + \Delta_a + \Delta_{v\theta} \\ &= 0.190 \text{ in.} + 0.0625 \text{ in.} + 0.0172 \text{ in.} \\ &= 0.270 \text{ in.} \end{aligned}$$

Recall that Δ_s is the vertical deflection of the spandrel beam due to live loads and superimposed dead loads. It accounts for the majority of the vertical deflection of the shelf angle.

The lateral deflection due to moment at the bottom of the hanger (see Figure 7-19) is,

$$\begin{aligned} \Delta_{hm} &= \frac{M_{net} L_c^2}{2EI_y} \\ &= \frac{8.64 \text{ kip-in.}(21.5 \text{ in.})^2}{2(29{,}000 \text{ ksi})(4.32 \text{ in.})} \\ &= 0.0159 \text{ in.} \end{aligned}$$

The lateral deflection due to the wind load at the bottom of the hanger (see Figure 7-20) is,

$$\Delta_{hw} = \frac{P_w L_c^3}{3EI_y}$$

$$= \frac{0.506 \text{ kip}(21.5 \text{ in.})^3}{3(29{,}000 \text{ ksi})(4.32 \text{ in.})}$$

$$= 0.0134 \text{ in.}$$

The total horizontal deflection at the bottom of the shelf angle is,

$$\Delta_h = \Delta_{hm} + \Delta_{hw}$$

$$= 0.0159 \text{ in.} + 0.0134 \text{ in.}$$

$$= 0.0293 \text{ in.}$$

This deflection is small and, therefore, neglected in the remainder of this example.

Consider roll beams at 9 ft on center in place of kickers. Without kickers to brace the bottom flange of the spandrel, torsion on the spandrel beam between the roll beams must be considered. Using the "flexural analogy," a relatively simple, conservative method is presented for calculating the additional vertical deflection of the roll beams due to twist of the spandrel. The hangers are located at the third-points along the torsionally unbraced length. For simplicity, the rotation of the roll beams will be neglected. See Figures 7-21 and 7-22.

The spandrel beam span for torsion is,

$$L_T = 9 \text{ ft}$$

The torsion imposed on the spandrel beam due to the brick at each hanger, neglecting the self-weight of the hangers, brackets, and shelf angles, is,

$$T_b = P_b\left(l_{eos} + l_{tol} + e_b\right)$$

$$= 1.68 \text{ kips}\left(11 \text{ in.} + 2 \text{ in.} + 4\tfrac{1}{8} \text{ in.}\right)$$

$$= 28.8 \text{ kip-in.}$$

The torsion imposed on the spandrel beam due to the stud back-up wall at each hanger is,

$$T_s = P_s\left(l_{eos} + l_{tol} + \frac{d_s}{2}\right)$$

$$= 0.104 \text{ kip}\left[11 \text{ in.} + 2 \text{ in.} + \frac{6 \text{ in.}}{2}\right]$$

$$= 1.66 \text{ kip-in.}$$

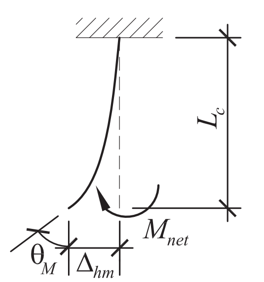


Fig. 7-19. Lateral deflection of hanger due to moment.

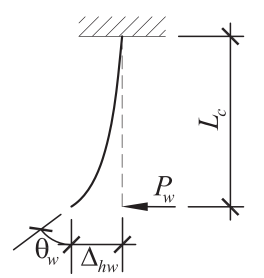


Fig. 7-20. Lateral deflection of hanger due to wind load.

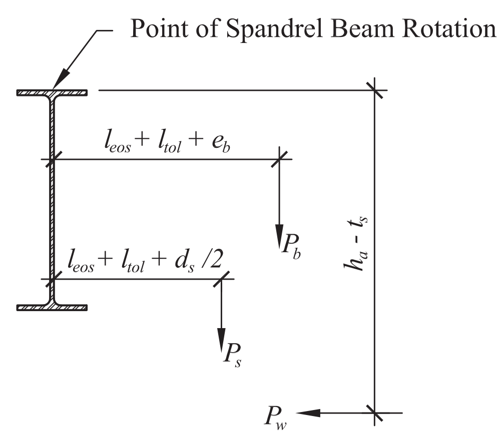


Fig. 7-21. Torsional loads on spandrel beam.

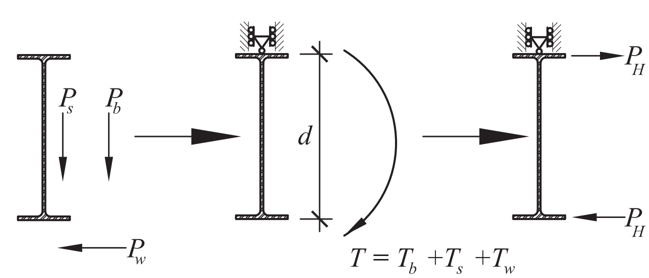


Fig. 7-22. Equivalent lateral forces on spandrel beam flange.

The torsion imposed on the spandrel beam due to the wind load at each hanger is,

$$\begin{aligned} T_w &= P_w\left(h_a - t_s\right) \\ &= 0.506 \text{ kip}\left[4 \text{ ft}\left(12 \text{ in./ft}\right) - 6\tfrac{1}{4} \text{ in.}\right] \\ &= 21.1 \text{ kip-in.} \end{aligned}$$

The torsion at each end of the span, assuming rotation about the spandrel top flange, is,

$$\begin{aligned} T &= T_b + T_s + T_w \\ &= 28.8 \text{ kip-in.} + 1.66 \text{ kip-in.} + 21.1 \text{ kip-in.} \\ &= 51.6 \text{ kip-in.} \end{aligned}$$

The equivalent force to apply to the bottom flange is,

$$\begin{aligned} P_H &= \frac{T}{d} \\ &= \frac{51.6 \text{ kip-in.}}{20.8 \text{ in.}} \\ &= 2.48 \text{ kips} \end{aligned}$$

The moment of inertia of the spandrel beam bottom flange is,

$$\begin{aligned} I_{bf} &= \frac{t_f b_f^3}{12} \\ &= \frac{0.535 \text{ in.}\left(6.53 \text{ in.}\right)^3}{12} \\ &= 12.4 \text{ in.}^4 \end{aligned}$$

The lateral displacement of the spandrel beam bottom flange at the hanger location is,

$$\begin{aligned} \Delta_{bf} &= \frac{5P_H L_T^3}{162EI_{bf}} \\ &= \frac{5\left(2.48 \text{ kip}\right)\left(9 \text{ ft}\right)^3\left(12 \text{ in./ft}\right)^3}{162\left(29{,}000 \text{ ksi}\right)\left(12.4 \text{ in.}^4\right)} \\ &= 0.268 \text{ in.} \end{aligned}$$

This deflection calculation assumes that the bottom flange of the spandrel consists of a single span that is simply supported at the roll beams. In reality, the bottom flange is continuous through this point, and the actual deflection would be somewhat less than determined here. If desirable, this continuity can be considered in the calculation.

The equivalent rotation at the top flange is,

$$\begin{aligned} \theta_{FA} &= \sin^{-1}\left(\frac{\Delta_{bf}}{d}\right) \\ &= \sin^{-1}\left(\frac{0.268 \text{ in.}}{20.8 \text{ in.}}\right) \\ &= 0.738^\circ \end{aligned}$$

The initial horizontal and vertical distances between the center of the top flange of spandrel rotation and the bottom of the shelf angle is,

$$\begin{aligned} x_i &= l_{eos} + l_{tol} + l_{ah} \\ &= 11 \text{ in.} + 2 \text{ in.} + 6 \text{ in.} \\ &= 19.0 \text{ in.} \end{aligned}$$

$$\begin{aligned} y_i &= h_a - t_s \\ &= 4 \text{ ft}\left(12 \text{ in./ft}\right) - 6\tfrac{1}{4} \text{ in.} \\ &= 41.8 \text{ in.} \end{aligned}$$

The distance between the point of rotation and the tip of the shelf angle is,

$$\begin{aligned} r &= \sqrt{x_i^2 + y_i^2} \\ &= \sqrt{\left(19.0 \text{ in.}\right)^2 + \left(41.8 \text{ in.}\right)^2} \\ &= 45.9 \text{ in.} \end{aligned}$$

The initial angle between the point of hanger rotation and the tip of the shelf angle with respect to vertical is,

$$\begin{aligned} \alpha &= \tan^{-1}\left(\frac{x_i}{y_i}\right) \\ &= \tan^{-1}\left(\frac{19.0 \text{ in.}}{41.8 \text{ in.}}\right) \\ &= 24.4^\circ \end{aligned}$$

The final angle between the point of hanger rotation and the tip of the shelf angle due to lateral translation of the shelf angle is,

$$\begin{aligned} \beta &= \alpha - \theta_{FA} \\ &= 24.4^\circ - 0.738^\circ \\ &= 23.7^\circ \end{aligned}$$

The final position of the tip of the shelf angle with respect to the point of hanger rotation is,

$$\begin{aligned} x_f &= r\sin(\beta) \\ &= (45.9\text{ in.})\sin(23.7°) \\ &= 18.4\text{ in.} \end{aligned}$$

$$\begin{aligned} y_f &= r\cos(\beta) \\ &= (45.9\text{ in.})\cos(23.7°) \\ &= 42.0\text{ in.} \end{aligned}$$

The vertical and horizontal rigid body translation of the tip of the shelf angle due to rotation of the spandrel beam and lateral translation of the hanger is,

$$\begin{aligned} \Delta_{vrb} &= y_f - y_i \\ &= 42.0\text{ in.} - 41.8\text{ in.} \\ &= 0.200\text{ in.} \end{aligned}$$

$$\begin{aligned} \Delta_{hrb} &= x_i - x_f \\ &= 19.0\text{ in.} - 18.4\text{ in.} \\ &= 0.600\text{ in.} \end{aligned}$$

The total vertical deflection of the shelf angle (see Figure 7-23) is,

$$\begin{aligned} \Delta_v &= \Delta_s + \Delta_a + \Delta_{v\theta} + \Delta_{vrb} \\ &= 0.190\text{ in.} + 0.0625\text{ in.} + 0.0172\text{ in.} + 0.200\text{ in.} \\ &= 0.470\text{ in.} \end{aligned}$$

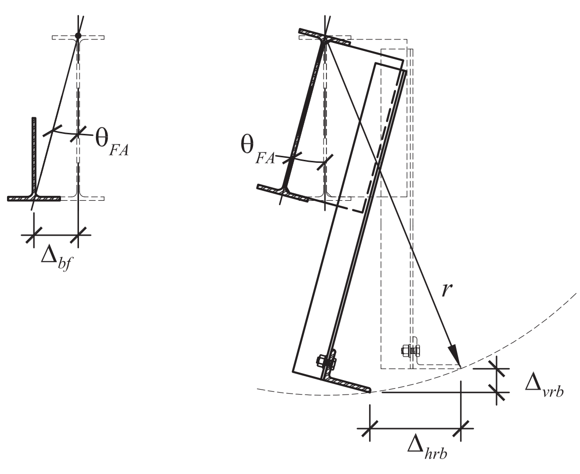


Fig. 7-23. Lateral translation of shelf angle.

The ratio of the vertical deflection to beam span is,

$$\begin{aligned} \frac{L}{\Delta_v} &= \frac{27\text{ ft}(12\text{ in./ft})}{0.470\text{ in.}} \\ &= 689 \end{aligned}$$

The total horizontal deflection of the shelf angle is,

$$\begin{aligned} \Delta_h &= \Delta_{hm} + \Delta_{hw} + \Delta_{hrb} \\ &= 0.0159\text{ in.} + 0.0134\text{ in.} + 0.600\text{ in.} \\ &= 0.629\text{ in.} \end{aligned}$$

The ratio of the horizontal deflection to the hanger length is,

$$\begin{aligned} \frac{h_a - t_s}{\Delta_h} &= \frac{4\text{ ft}(12\text{ in./ft}) - 6.25\text{ in.}}{0.629\text{ in.}} \\ &= 66.4 \end{aligned}$$

Comments:
Additional cladding support design steps, such as the following, are not included in this example:

- Design of the angle hanger for combined flexure and tension, considering yielding, lateral-torsional buckling, and leg local buckling for bending about the principal axes.
- Design of the field welds between the shelf angle and the hanger.
- Design of the bracket plate, including the welds to the hanger and spandrel beam.
- Design of the kicker connection to the bottom flange.

This example shows that the designer may need to account for rotational deflections due to the flexural stiffness of a shelf angle hanger when calculating shelf angle deflections. Tide and Krogstad (1993) assume the tip of the shelf angle deflects and rotates downward, shifting the point of load of the relatively rigid brick away from the tip of the angle. They suggest that the brick load should be assumed to act on the angle at ½ in. outboard of the interior face of the brick.

For the case where kickers brace the bottom flange of the spandrel beam against twist at each hanger, the relative contributions of each deflection component are summarized here.

Deflection of shelf angle alone

$$\frac{\Delta_a}{\Delta} = \frac{0.0625\text{ in.}}{0.270\text{ in.}} = 23.2\%$$

Rotation of bottom of shelf angle

$$\frac{\Delta_{v\theta}}{\Delta} = \frac{0.0172 \text{ in.}}{0.270 \text{ in.}} = 6.4\%$$

Deflection of spandrel beam alone

$$\frac{\Delta_s}{\Delta} = \frac{0.190 \text{ in.}}{0.270 \text{ in.}} = 70.4\%$$

Where roll beams brace the spandrel beam against twist at each hanger, the relative contributions of each deflection component are summarized here.

Deflection of shelf angle alone

$$\frac{\Delta_a}{\Delta_v} = \frac{0.0625 \text{ in.}}{0.470 \text{ in.}} = 13.3\%$$

Rotation of bottom of shelf angle

$$\frac{\Delta_{v\theta}}{\Delta_v} = \frac{0.0172 \text{ in.}}{0.470 \text{ in.}} = 3.7\%$$

Deflection of spandrel beam alone

$$\frac{\Delta_s}{\Delta_v} = \frac{0.190 \text{ in.}}{0.469 \text{ in.}} = 40.4\%$$

Rigid-body rotation

$$\frac{\Delta_{vrb}}{\Delta_v} = \frac{0.200 \text{ in.}}{0.470 \text{ in.}} = 42.6\%$$

Although the calculated vertical deflection is less than $L/600$, it is still likely too large—assuming 50% compressibility of the sealant material, the horizontal sealant joint beneath the angle would have to be more than 1 in. wide to accommodate both the structural movement and the brick expansion.

The lateral translation of the shelf angle without kickers is also of concern.

Translation due to concentrated moment at bottom of hanger $\quad \dfrac{\Delta_{hm}}{\Delta_h} = \dfrac{0.0159 \text{ in.}}{0.629 \text{ in.}} = 2.5\%$

Translation due to lateral wind force at bottom of hanger $\quad \dfrac{\Delta_{hw}}{\Delta_h} = \dfrac{0.0134 \text{ in.}}{0.629 \text{ in.}} = 2.1\%$

Translation due to rigid body motion of spandrel $\quad \dfrac{\Delta_{hrb}}{\Delta_h} = \dfrac{0.600 \text{ in.}}{0.629 \text{ in.}} = 95.4\%$

While the floor slab may provide some torsional restraint to the beam (and reduce Δ_{hrb}), the calculated horizontal displacement is significantly larger than $L/600$. If the bottom flange of the spandrel beam in this case is not laterally braced between the roll beams, a stiffer spandrel beam is required.

Example 7.4—Shelf Angle for Brick Veneer Supported by Long Hanger System on Floor Spandrel Beam

For the clay brick veneer supported at the first elevated floor level of a building by a shelf angle as illustrated in Figure 7-24, determine the vertical and horizontal deflections at the tip of the shelf angle assuming the kicker is:

(a) pin-connected to the hanger

(b) rigidly connected to the hanger

Given:

The supported height of the brick, h = 18 ft. The slab thickness, t_s = 6¼ in. The distance between the top of the slab and the bottom of the angle, h_a = 8 ft. The distance between the top of the first floor slab and the top of the foundation below, h_s = 18 ft.

The wall is subjected to a "components and cladding" wind pressure, p_w = 25 psf acting toward the building. The weight of the brick veneer, w_b = 40 psf. The weight of the

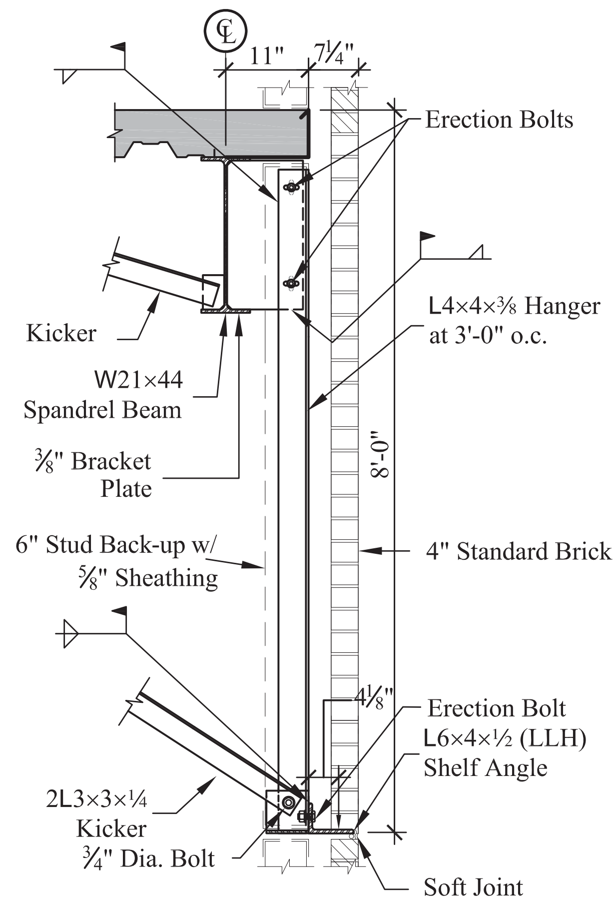


Fig. 7-24. Section of spandrel beam with long hanger system.

stud backup wall system, w_s = 10 psf. The depth of the stud wall assembly is d_s = 6 in. Note that the weight of the shelf angle is neglected since its load causes the beam to deflect before the brick is installed.

The shelf angle is an L6×4×½, the horizontal leg dimension, l_{ah} = 6 in. The tolerance is l_{tol} = 2 in. From Table 7-5, the maximum deflection of the shelf angle is Δ_a = 0.0348 in.

Note that an angle with a minimum 4-in. leg perpendicular to the façade is recommended to facilitate adjustability with respect to the façade. The L4×4×⅜ hanger angle has the following properties:

$$A = 2.86 \text{ in.}^2$$
$$I_x = 4.32 \text{ in.}^4$$
$$\bar{x} = \bar{y} = 1.13 \text{ in.}$$
$$s = 3 \text{ ft on center}$$

The 2L3×3×¼ kicker angle has the following properties:

$$A = 2.87 \text{ in.}^2$$
$$I_x = 2.46 \text{ in.}^4$$
$$\alpha_k = 45°$$

The W21×44 spandrel beam has the following properties:

$$d = 20.7 \text{ in.}$$
$$t_f = 0.450 \text{ in.}$$

Under superimposed dead load and live load effects, the spandrel has deflection, Δ_s = 0.216 in.

The distance from the spandrel centerline to the edge of the slab, l_{eos} = 11 in.

As in Example 7.3, assume that the brick load is applied to the relieving angle ½-in. outboard of the interior face of the brick. Thus, the distance from the back of the vertical leg of the shelf angle to the point of the brick load, e_b = 4⅛ in.

Solution (a): Kicker Pin-Connected to the Hanger

See Figure 7-25. The unbraced length of the hanger below the bracket is,

$$\begin{aligned} L &= h_a - \left(d + t_s\right) + t_f \\ &= 8 \text{ ft}\left(12 \text{ in./ft}\right) - \left(20.7 \text{ in.} + 6\tfrac{1}{4} \text{ in.}\right) + 0.450 \text{ in.} \\ &= 69.5 \text{ in.} \end{aligned}$$

The length of the kicker is,

$$\begin{aligned} L_k &= \frac{h_a - t_s}{\cos\left(\alpha_k\right)} \\ &= \frac{8 \text{ ft}\left(12 \text{ in./ft}\right) - 6\tfrac{1}{4} \text{ in.}}{\cos\left(45°\right)} \\ &= 127 \text{ in.} \end{aligned}$$

In this case, the kicker resists only axial loads.

The weight of brick per hanger is,

$$\begin{aligned} P_b &= w_b h s \\ &= 0.040 \text{ kip/ft}^2\left(18 \text{ ft}\right)\left(3 \text{ ft}\right) \\ &= 2.16 \text{ kips} \end{aligned}$$

The height of the stud wall supported on the hanger is,

$$\begin{aligned} h_{st} &= h_a - t_s \\ &= 8 \text{ ft} - \left(\frac{6\tfrac{1}{4} \text{ in}}{12 \text{ in./ft}}\right) \\ &= 7.48 \text{ ft} \end{aligned}$$

The weight of the stud wall supported on each hanger is,

$$\begin{aligned} P_s &= w_s h_{st} s \\ &= 0.010 \text{ kip/ft}^2\left(7.48 \text{ ft}\right)\left(3 \text{ ft}\right) \\ &= 0.224 \text{ kip} \end{aligned}$$

The lateral wind load applied at the bottom of each hanger is,

$$\begin{aligned} P_w &= p_w\left(\frac{h_s - h_a}{2} + \frac{h_{st}}{2}\right)s \\ &= 0.025 \text{ kip/ft}^2\left(\frac{18 \text{ ft} - 8 \text{ ft}}{2} + \frac{7.48 \text{ ft}}{2}\right)\left(3 \text{ ft}\right) \\ &= 0.656 \text{ kip} \end{aligned}$$

The hanger is fixed for flexure at the bottom of the bracket plate and pinned for flexure where it attaches to the kicker. Also, the hanger rotates about its centroid. The moment

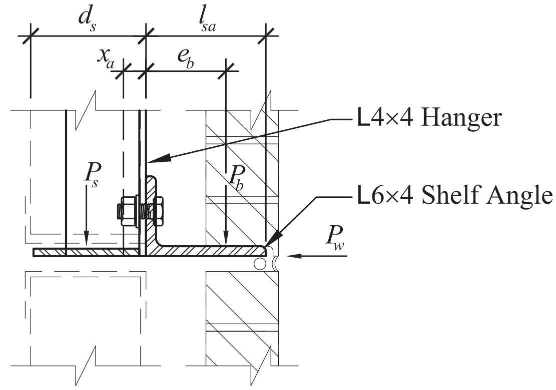


Fig. 7-25. Loads on hanger.

at the bottom of the hanger due to the eccentricity of the brick is,

$$M_b = P_b\left(e_b + \bar{x}\right)$$
$$= 2.16 \text{ kips}\left(4\tfrac{1}{8} \text{ in.} + 1.13 \text{ in.}\right)$$
$$= 11.4 \text{ kip-in.}$$

The moment at the bottom of the hanger due to the eccentricity of the stud wall is,

$$M_s = P_s\left(\frac{d_s}{2} - \bar{x}\right)$$
$$= 0.224 \text{ kip}\left(\frac{6 \text{ in.}}{2} - 1.13 \text{ in.}\right)$$
$$= 0.419 \text{ kip-in.}$$

The net moment at the bottom of the hanger is,

$$M_{net} = M_b - M_s$$
$$= 11.4 \text{ kip-in.} - 0.419 \text{ kip-in.}$$
$$= 11.0 \text{ kip-in.}$$

The flexural stiffness of the hanger is,

$$K_h = \frac{4EI_x}{L}$$
$$= \frac{4\left(29{,}000 \text{ ksi}\right)\left(4.32 \text{ in.}^4\right)}{69.5 \text{ in.}}$$
$$= 7{,}210 \text{ kip-in./rad.}$$

The rotation at the bottom of the hanger due to M_{net} (see Figure 7-26) is,

$$\theta_h = \frac{M_{net}}{K_h}$$
$$= \frac{11.0 \text{ kip-in.}}{7{,}210 \text{ kip-in./rad.}}$$
$$= 0.00153 \text{ rad. (or } 0.0874°)$$

The initial horizontal distance between the hanger centroid and the tip of the shelf angle is,

$$x_i = \bar{x} + l_{ah}$$
$$= 1.13 \text{ in.} + 6 \text{ in.}$$
$$= 7.13 \text{ in.}$$

The vertical displacement of the shelf angle due to rotation at the bottom of the hanger is,

$$\Delta_\theta = x_i \sin\theta_h$$
$$= \left(7.13 \text{ in.}\right)\sin\left(0.0874°\right)$$
$$= 0.0109 \text{ in.}$$

This deflection is very small, but has been included in the total deflection for completeness. In practice, this deflection is sufficiently small that it may, in most cases, be neglected.

The elongation of the hanger due to the axial load is,

$$\Delta_e = \frac{\left(P_b + P_s\right)L}{AE}$$
$$= \frac{\left(2.16 \text{ kips} + 0.224 \text{ kip}\right)\left(69.5 \text{ in.}\right)}{2.86 \text{ in.}^2\left(29{,}000 \text{ ksi}\right)}$$
$$= 0.00200 \text{ in.}$$

The total vertical deflection at the tip of the angle (neglecting the deflection due to hanger elongation, which is small) is,

$$\Delta_v = \Delta_a + \Delta_s + \Delta_\theta$$
$$= 0.0348 \text{ in.} + 0.216 \text{ in.} + 0.0109 \text{ in.}$$
$$= 0.262 \text{ in.}$$

The lateral deflection due to the axial load in the kicker is,

$$\Delta_h = \frac{P_w\left(h_a - t_s\right)}{AE\cos^2\left(\alpha_k\right)\sin\left(\alpha_k\right)}$$
$$= \frac{0.656 \text{ kip}\left[8 \text{ ft}\left(12 \text{ in./ft}\right) - 6\tfrac{1}{4} \text{ in.}\right]}{2.87 \text{ in.}^2\left(29{,}000 \text{ ksi}\right)\cos^2\left(45°\right)\sin\left(45°\right)}$$
$$= 0.00200 \text{ in.}$$

This deflection is very small, and is typically neglected in design.

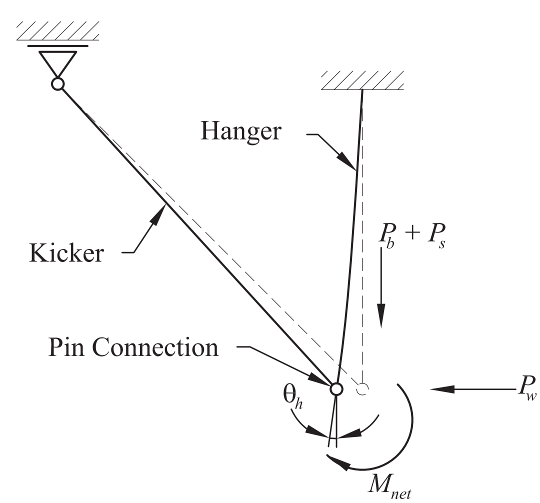


Fig. 7-26. Rotation of hanger with kicker pinned for flexure.

The relative contributions of each deflection to the total are:

Shelf angle $\dfrac{\Delta_a}{\Delta_v} = 9.9\%$

Spandrel beam $\dfrac{\Delta_s}{\Delta_v} = 84.8\%$

Hanger rotation $\dfrac{\Delta_\theta}{\Delta_v} = 4.4\%$

Hanger elongation $\dfrac{\Delta_e}{\Delta_v} = 0.9\%$

Solution (b): Kicker Rigidly Connected to the Hanger

In this case, the kicker provides flexural resistance. To determine the rotation of the shelf angle, the hanger and kicker can be treated as springs acting in parallel (see Figure 7-27). The hanger flexural stiffness, assuming that the top of the hanger is fixed for flexure at the bracket plate, is,

$$
\begin{aligned}
K_h &= \frac{4EI_x}{L} \\
&= \frac{4\left(29{,}000 \text{ ksi}\right)\left(4.32 \text{ in.}^4\right)}{69.5 \text{ in.}} \\
&= 7{,}210 \text{ kip-in./rad.}
\end{aligned}
$$

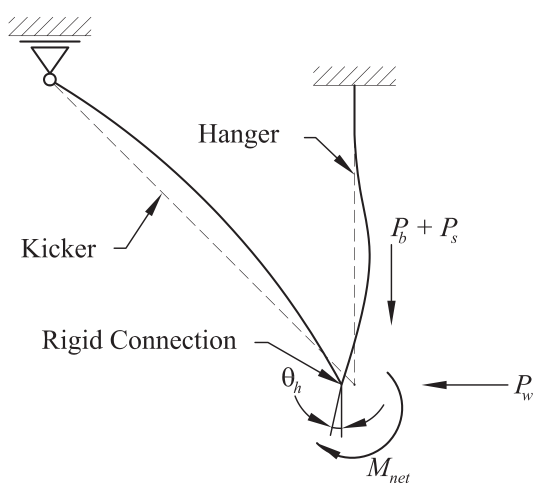


Fig. 7-27. Rotation of hanger with kicker fixed for rotation.

The kicker flexural stiffness, assuming that the top end of the kicker is pinned for flexure, is,

$$
\begin{aligned}
K_k &= \frac{3EI_x}{L_k} \\
&= \frac{3\left(29{,}000 \text{ ksi}\right)\left(2.46 \text{ in.}^4\right)}{127 \text{ in.}} \\
&= 1{,}690 \text{ kip-in./rad.}
\end{aligned}
$$

The rotation at the centroid of the hanger is,

$$
\begin{aligned}
\theta &= \frac{M_{net}}{K_k + K_h} \\
&= \frac{11.0 \text{ kip-in.}}{1{,}690 \text{ kip-in./rad.} + 7{,}210 \text{ kip-in./rad.}} \\
&= 0.00124 \text{ rad. (or } 0.0710°)
\end{aligned}
$$

The vertical displacement of the shelf angle due to rotation at the bottom of the hanger is,

$$
\begin{aligned}
\Delta_\theta &= x_i \sin\left(\theta\right) \\
&= \left(7.13 \text{ in.}\right)\sin\left(0.0710°\right) \\
&= 0.00884 \text{ in.}
\end{aligned}
$$

This deflection is very small and will be neglected in the calculation of the total vertical deflection. The total vertical deflection at the tip of the angle is,

$$
\begin{aligned}
\Delta_v &= \Delta_a + \Delta_s \\
&= 0.035 \text{ in.} + 0.216 \text{ in.} \\
&= 0.251 \text{ in.}
\end{aligned}
$$

Note that fixing the kicker for rotation at the shelf angle reduces the vertical deflection of the system, but only very slightly. In practice, the cost of developing the flexural strength of the connection probably is not justified by this small increase in stiffness.

Comments:

Additional cladding support design steps such as the following are not included in this example:

- Design of the angle hanger for combined flexure and tension, considering yielding, lateral-torsional buckling, and leg local buckling for bending about the principal axes.
- Design of the field welds between the shelf angle and the hanger.
- Design of the bracket plate, including the welds to the hanger and spandrel beam.

- Design of the kicker for combined flexure and compression, considering yielding, lateral-torsional buckling, and leg local buckling for bending about the principal axes.
- Design of the kicker connection to the bottom flange.

In this example the bracket plate is assumed to be rigidly attached to the spandrel beam, and the kickers bracing the bottom flange of the spandrel beam are assumed to resist the associated torsion. If kickers to the bottom flange of the beam are impractical or not possible, the designer must check the spandrel beam for torsion. Torsion on a wide-flange spandrel such as this one is likely to lead to substantial vertical deflections of the shelf angle. As in previous examples, roll beams can be provided to help resist this torsion. If the attachment of the hanger to the spandrel beam can be detailed so as to impart no moment to the spandrel (i.e., a hinge), torsion in the spandrel can be eliminated, but the hanger must be designed for reverse-curvature bending. See Figures 7-28 and 7-29.

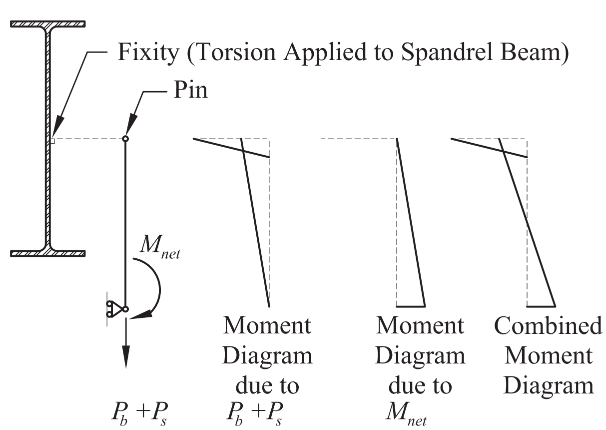


Fig. 7-28. Hanger assembly moment diagram with pin at spandrel beam web.

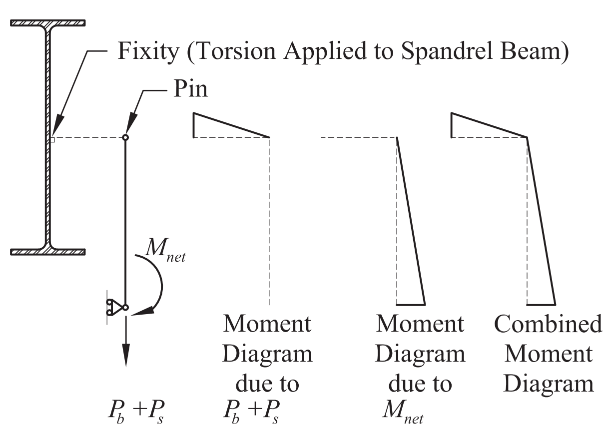


Fig. 7-29. Hanger assembly moment diagram with fixity at spandrel beam web.

Example 7.5—Shelf Angle for Brick Veneer Supported by Slab Edge

For the clay brick veneer illustrated in Figure 7-30,

1. Select the angle leg thickness required to support the brick cladding where the maximum vertical deflection of the horizontal leg is 1⁄16 in.
2. Design the field weld between the top edge of the angle and the pour stop.
3. Select the headed studs to transfer loads from the pour stop into the slab.
4. Design the slab reinforcement to support the cladding and a 6-in. stud back-up wall assembly.

Given:
For this example, assume that the following load combinations control:

Shelf Angle and Stud
Strength: $1.4D$
Deflection: D

Slab
Strength: $1.2D + 1.6L + 0.8W$

Use an L6×4.

The story height, h = 14 ft. The slab thickness, t_s = 7½ in. The floor construction consists of 4½ in. of 4,000-psi normal-weight concrete on a 3-in. metal floor deck parallel to the spandrel (total thickness = 7½ in.). The reinforcing steel yield strength, f_y = 60 ksi. The thickness of the slab above the flutes, t_c = 4½ in., and the slab bar clear cover, c_c = 1 in. The edge of the slab dimension, l_{eos} = 11 in.

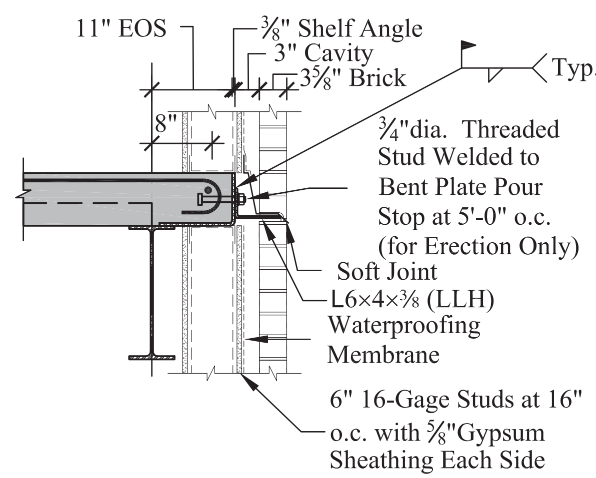


Fig. 7-30. Section of slab edge with shelf angle supporting brick veneer.

The weight of the brick veneer, w_b = 40 psf and its thickness, t_m = 3⅝ in. The cavity width between the exterior face of the vertical leg of the shelf angle and the interior face of the brick, w_c = 3 in.

The weight of the stud back-up wall including sheathing, waterproofing, and insulation, w_s = 10 psf. The eccentricity of the stud back-up with respect to the spandrel centerline, e_s = 8 in. The stud backup resists a components and cladding wind suction, p_w = 25 psf acting away from the building.

Solution 1. Select the angle thickness:

Unlike in the previous examples where the shelf angle was supported at discrete locations along the length at hangers, here the shelf angle is supported continuously by welds to the pour stop. See Figure 7-31.

The weight of brick per foot is,

$$\begin{aligned} P_b &= w_b h \\ &= 0.040 \text{ kip/ft}^2 \left(14 \text{ ft}\right) \\ &= 0.560 \text{ kip/ft} \end{aligned}$$

Try a ⅜-in. angle thickness. The moment of inertia of the angle leg per foot of length is,

$$\begin{aligned} I_a &= \frac{bt^3}{12} \\ &= \frac{\left(12 \text{ in./ft}\right)\left(\tfrac{3}{8} \text{ in.}\right)^3}{12} \\ &= 0.0527 \text{ in.}^4\text{/ft} \end{aligned}$$

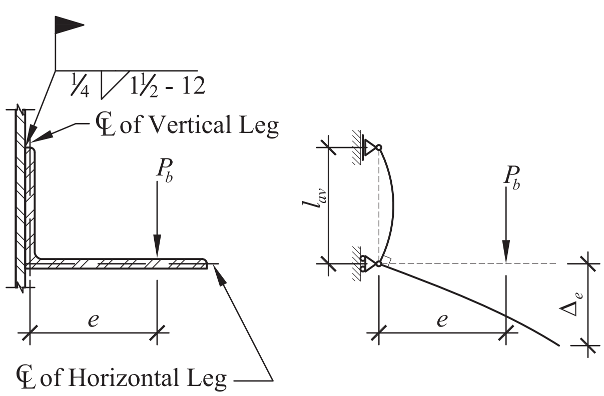


Fig. 7-31. Analysis model of shelf angle.

The elastic section modulus of the angle leg per foot of length is,

$$\begin{aligned} S_a &= \frac{bt^2}{6} \\ &= \frac{\left(12 \text{ in./ft}\right)\left(\tfrac{3}{8} \text{ in.}\right)^2}{6} \\ &= 0.281 \text{ in.}^3\text{/ft} \end{aligned}$$

The plastic section modulus of the angle leg per foot of length is,

$$\begin{aligned} Z_a &= \frac{bt^2}{4} \\ &= \frac{\left(12 \text{ in./ft}\right)\left(\tfrac{3}{8} \text{ in.}\right)^2}{4} \\ &= 0.422 \text{ in.}^3\text{/ft} \end{aligned}$$

Tide and Krogstad (1993) suggest that the brick should be considered to load the horizontal leg of the shelf angle ½ in. outboard of the interior face of the brick. Basing the analysis of the shelf angle on centerline dimensions, the eccentricity of the brick load with respect to the centerline of the vertical leg of the angle is,

$$\begin{aligned} e &= \frac{t}{2} + w_c + \tfrac{1}{2} \text{ in.} \\ &= \frac{\tfrac{3}{8} \text{ in.}}{2} + 3 \text{ in.} + \tfrac{1}{2} \text{ in.} \\ &= 3.69 \text{ in.} \end{aligned}$$

The deflection at the top of the horizontal leg treating the leg as a rigid cantilever is,

$$\begin{aligned} \Delta_c &= \frac{P_b e^2}{6EI_a}\left(3l_{ah} - e\right) \\ &= \frac{0.560 \text{ kip/ft}\left(3.69 \text{ in.}\right)^2}{6\left(29{,}000 \text{ ksi}\right)\left(0.0527 \text{ in.}^4\text{/ft}\right)}\left[3\left(6 \text{ in.}\right) - 3.69 \text{ in.}\right] \\ &= 0.0119 \text{ in.} \end{aligned}$$

The stiffening effect of the fillet is conservatively neglected in this calculation.

The moment on the vertical leg is,

$$\begin{aligned} M_a &= P_b e \\ &= 0.560 \text{ kip/ft}\left(3.69 \text{ in.}\right) \\ &= 2.07 \text{ kip-in./ft} \end{aligned}$$

The rotation due to the moment on the vertical leg is,

$$\begin{aligned}\theta_a &= \frac{M_a l_{av}}{3EI_a}\\ &= \frac{2.07 \text{ kip-in./ft}\left(4 \text{ in.}\right)}{3\left(29{,}000 \text{ ksi}\right)\left(0.0527 \text{ in.}^4\text{/ft}\right)}\\ &= 0.00181 \text{ rad. (or } 0.104^\circ)\end{aligned}$$

The rigid body rotation of the horizontal leg is,

$$\begin{aligned}\Delta_{rb} &= \left(l_{ah} - \frac{t}{2}\right)\sin\theta_a\\ &= \left(6 \text{ in.} - \frac{3/8 \text{ in.}}{2}\right)\sin\left(0.104^\circ\right)\\ &= 0.0106 \text{ in.}\end{aligned}$$

The total vertical deflection of the shelf angle is,

$$\begin{aligned}\Delta_t &= \Delta_c + \Delta_{rb}\\ &= 0.0119 \text{ in.} + 0.0106 \text{ in.}\\ &= 0.0225 \text{ in.} \le 1/16 \text{ in. } \textbf{o.k.}\end{aligned}$$

For flexure on the horizontal leg of the angle, the critical section is at the toe of the fillet (a distance k from the back of the angle).

$$k = 7/8 \text{ in.}$$

The bending length is,

$$\begin{aligned}l_b &= (t + w_c + 1/2 \text{ in.}) - k\\ &= (3/8 \text{ in.} + 3 \text{ in.} + 1/2 \text{ in.}) - 7/8 \text{ in.}\\ &= 3.00 \text{ in.}\end{aligned}$$

The required flexural strength is,

$$\begin{aligned}M_u &= 1.4P_b l_b\\ &= 1.4\left(0.560 \text{ kip/ft}\right)\left(3.00 \text{ in.}\right)\\ &= 2.35 \text{ kip-in./ft}\end{aligned}$$

The available flexural strength is,

$$\begin{aligned}\phi_b M_n &= 0.90F_y Z_a \le 0.90\left(1.6F_y S_a\right)\\ &= 0.90(36 \text{ ksi})(0.422 \text{ in.}^3\text{/ft})\\ &\qquad \le 0.90(1.6)(36 \text{ ksi})(0.281 \text{ in.}^3\text{/ft})\\ &= 13.7 \text{ kip-in./ft} \le 14.6 \text{ kip-in./ft}\\ &= 13.7 \text{ kip-in./ft} \ge M_u \quad \textbf{o.k.}\end{aligned}$$

Use an L6×4×3/8.

Solution 2. Design the field weld between the top edge of the angle and the pour stop:

Try an intermittent 1/4-in. fillet weld with F_{EXX} = 70 ksi. Note that the weld is only required along the top edge of the angle and that no weld is required on the bottom edge because the bottom edge is bearing against the pour stop. This has the added benefit of requiring only downhand field welds. It may be necessary to mask the top edge of the angle during galvanizing because the zinc from the galvanizing process can compromise the effectiveness of the weld. The designer should determine whether the top edge of the angle and weld-affected zones should be touched-up with zinc-rich primer.

The factored vertical shear along the angle is,

$$\begin{aligned}V_u &= 1.4P_b\\ &= 1.4\left(0.560 \text{ kip/ft}\right)\\ &= 0.784 \text{ kip/ft}\end{aligned}$$

Using the angle leg dimensions as l_{ab} = 6 in. and l_{av} = 4 in., the factored horizontal tension force acting outward on the weld is,

$$\begin{aligned}H_u &= V_u\left(\frac{l_{ah} - 4/9\, t_m}{l_{av}}\right)\\ &= 0.784 \text{ kip/ft}\left(\frac{6 \text{ in.} - 4/9\left(3\tfrac{5}{8} \text{ in.}\right)}{4 \text{ in.}}\right)\\ &= 0.860 \text{ kip/ft}\end{aligned}$$

The resultant shear on the weld is,

$$\begin{aligned}F_u &= \sqrt{V_u^2 + H_u^2}\\ &= \sqrt{\left(0.784 \text{ kip/ft}\right)^2 + \left(0.860 \text{ kip/ft}\right)^2}\\ &= 1.16 \text{ kips/ft}\end{aligned}$$

The angle between the load and the longitudinal axis of the weld is,

$$\theta_w = 90^\circ$$

The weld effective throat thickness is,

$$\begin{aligned}t_w &= 0.707w\\ &= 0.707(1/4 \text{ in.})\\ &= 0.177 \text{ in.}\end{aligned}$$

The fillet weld nominal strength per *Specification* Equation J2-5 is,

$$\begin{aligned} F_w &= 0.6F_{EXX}\left[1+0.5\sin^{1.5}\left(\theta_w\right)\right] \\ &= 0.6\left(70\text{ ksi}\right)\left[1+0.5\sin^{1.5}\left(90°\right)\right] \\ &= 63.0\text{ ksi} \end{aligned}$$

The minimum required length of fillet weld per foot of plate is,

$$\begin{aligned} l_w &= \frac{F_u}{\phi_v F_w t_w} \\ &= \frac{1.16\text{ kips/ft}}{0.75\left(63\text{ ksi}\right)\left(0.177\text{ in.}\right)} \\ &= 0.139\text{ in./ft} \end{aligned}$$

Per AISC Specification Section J2.2b, the minimum length for intermittent fillet welds is the larger of four times the nominal weld size and 1½ in. Thus, the 1½-in. minimum length applies in this case.

Use a ¼-in. intermittent fillet weld 1½-in.-long per foot along the length of the shelf angle.

Solution 3. Design the Headed Studs to Transfer Loads from the Pour Stop into the Slab:
In this case, the headed studs must resist vertical shear from the brick as well as the tension component of a couple that results from the eccentricity of the brick. Try 6-in.-long, ¾-in.-diameter studs along the slab edge.

With each stud placed at mid-depth of the slab, the length of the moment arm between forces in the couple is,

$$\begin{aligned} e_{Mu} &= \frac{t_s}{2} \\ &= \frac{7\tfrac{1}{2}\text{ in.}}{2} \\ &= 3.75\text{ in.} \end{aligned}$$

The factored tension force on the studs per foot of slab is,

$$\begin{aligned} N_u &= \frac{M_u}{e_{Mu}} \\ &= \frac{2.35\text{ kip-in./ft}}{3.75\text{ in.}} \\ &= 0.627\text{ kip/ft} \end{aligned}$$

The factored shear force on the studs per foot of slab is,

$$V_u = 0.784\text{ kip/ft}$$

Try a 2-ft stud spacing (s = 2 ft). Because the studs are farther apart than three times the edge distance (3¾ in.), group effects need not be considered and the studs can be considered to act separately. From Tables 5-12 and 5-13, the design tensile and shear strengths, ϕN_n and ϕV_n, respectively, for ¾-in.-diameter headed studs with 6-in. embedment into a 7½-in.-thick normal-weight concrete slab are as listed here.

$$\phi N_n = 5.37\text{ kips}$$
$$\phi V_n = 3.38\text{ kips}$$

The tension demand-to-capacity ratio is,

$$\begin{aligned} \frac{sN_u}{\phi N_n} &= \frac{2\text{ ft}\left(0.627\text{ kip/ft}\right)}{5.37\text{ kips}} \\ &= 0.234 \le 1.0 \end{aligned}$$

The shear demand-to-capacity ratio is,

$$\begin{aligned} \frac{sV_u}{\phi V_n} &= \frac{2\text{ ft}\left(0.784\text{ kip/ft}\right)}{3.38\text{ kip}} \\ &= 0.464 \le 1.0 \end{aligned}$$

Because these individual demand-to-capacity ratios are greater than 0.20, the interaction of tension and shear must be investigated following ACI 318-05 Equation D-31.

The shear-tension interaction is,

$$\begin{aligned} \frac{sN_u}{\phi N_n} + \frac{sV_u}{\phi V_n} &= 0.234 + 0.464 \\ &= 0.698 \le 1.0 \quad \textbf{o.k.} \end{aligned}$$

Use one 6-in.-long, ¾-in.-diameter stud at 2 ft on center.

Note that, in addition to the headed studs, threaded studs as shown in Figure 7-30 should also be provided. These studs are welded to the pour stop and are for erection alignment purposes only.

Solution 4. Design the Slab Reinforcement to Support the Cladding and the Stud Backup Wall:
The weight of the concrete is carried by the bent plate and need not be considered in the design of the slab reinforcement. Additionally, the floor live load moment on the slab cantilever is very small because most of the overhang is covered by the backup wall. In this case, this moment can be neglected.

The weight of the stud backup wall per foot of slab is,

$$\begin{aligned}P_s &= w_s h\\ &= 0.010\text{ kips/ft}^2\left(14\text{ ft}\right)\\ &= 0.140\text{ kip/ft}\end{aligned}$$

The factored moment at the centerline of the spandrel is,

$$\begin{aligned}M_u &= 1.2[P_b(l_{eos} + t + w_c + \tfrac{1}{2}\text{ in.}) + P_s e_s]\\ &= 1.2[0.560\text{ kip/ft }(11\text{ in.} + \tfrac{3}{8}\text{ in.} + 3\text{ in.}\\ &\quad + \tfrac{1}{2}\text{ in.}) + 0.140\text{ kip/ft }(8\text{ in.})]\\ &= 11.3\text{ kip-in./ft}\end{aligned}$$

The wind suction load on the slab edge is,

$$\begin{aligned}N_{uw} &= 0.8 p_w h\\ &= 0.8\left(0.025\text{ kip/ft}^2\right)\left(14\text{ ft}\right)\\ &= 0.280\text{ kip/ft}\end{aligned}$$

Try #4 reinforcing bars at 16 in. on center. Note that this bar spacing violates the maximum bar spacing for beams and slabs presented in ACI 318-05 Section 10.6.4. The ACI criterion is intended to limit surface cracks in concrete. In this example, it is assumed that the slab will be covered with a finished floor; therefore, surface cracks are not a concern. If surface cracks along the slab edges are a concern, reinforcing steel may be added in the form of additional bars that supplement the typical reinforcing steel. It is important, however, that enough of the steel be developed at the critical section.

From Table 5-5, the design flexural strength of the slab is as follows (assuming that the reinforcement is fully developed at the critical section):

$$\phi M_{ns} = 25.4\text{ kip-in./ft}$$

The area of steel provided is,

$$A_s = 0.150\text{ in.}^2\text{/ft}$$

The slab bar diameter is,

$$d_b = \tfrac{1}{2}\text{ in.}$$

The length of the hooked bar beyond the critical section of the slab (see Figure 7-32) is,

$$\begin{aligned}l_h &= l_{eos} - c_c - \frac{d_b}{2}\\ &= 11\text{ in.} - 1\text{ in.} - \frac{\tfrac{1}{2}\text{ in.}}{2}\\ &= 9.75\text{ in.}\end{aligned}$$

The development length of #4 hooked bar in 4,000-psi normal-weight concrete is,

$$l_{dh} = 10\text{ in.}$$

In this particular case, the length of the hook is slightly less than the development length, so the effective steel area must be reduced. Additionally, some of this steel area is required to resist out-of-plane tension loads on the slab edge due to wind loads and the eccentricity of the curtain wall. Hence, the nominal flexural strength of the slab must be recalculated.

The effective depth from the top of the deck to the reinforcing steel is,

$$\begin{aligned}d &= t_c - 1\text{ in.} - \frac{d_b}{2}\\ &= 4\tfrac{1}{2}\text{ in.} - 1\text{ in.} - \frac{\tfrac{1}{2}\text{ in.}}{2}\\ &= 3.25\text{ in.}\end{aligned}$$

The effective area of steel in the slab is,

$$\begin{aligned}A_{se} &= \frac{l_h}{l_{dh}}\left(A_s - \frac{N_{uw}}{\phi f_y}\right)\\ &= \frac{9.75\text{ in.}}{10\text{ in.}}\left(0.150\text{ in.}^2\text{/ft} - \frac{0.280\text{ kip/ft}}{0.90\left(60\text{ ksi}\right)}\right)\\ &= 0.141\text{ in.}^2\text{/ft}\end{aligned}$$

By strain compatibility and equilibrium calculations not shown here, $e_s > e_y$, the steel stress can be taken equal to f_y, and the concrete compression block depth is,

$$\begin{aligned}a &= \frac{A_{se} f_y}{0.85 b f'_c}\\ &= \frac{0.141\text{ in.}^2\left(60\text{ ksi}\right)}{0.85\left(12\text{ in.}\right)\left(4\text{ ksi}\right)}\\ &= 0.207\text{ in.}\end{aligned}$$

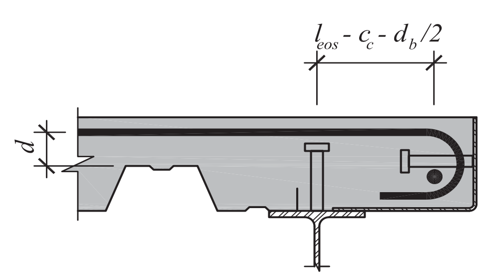


Fig. 7-32. Slab dimensions for analysis.

Thus,

$$\phi = 0.9\,(\text{since } \varepsilon_s > 0.005)$$

$$\begin{aligned}
\phi M_{ns} &= \phi A_{se} f_y \left(d - \frac{a}{2}\right) \\
&= 0.9\left(0.141 \text{ in.}^2\right)\left(60 \text{ ksi}\right)\left(3.25 \text{ in.} - \frac{0.207 \text{ in.}}{2}\right) \\
&= 24.0 \text{ kip-in./ft} > 11.3 \text{ kip-in./ft} \quad \textbf{o.k.}
\end{aligned}$$

Comments:

An alternative, less complicated approximation to the calculations shown above is as follows. The nominal flexural strength of the slab is,

$$\begin{aligned}
\frac{\phi M_{ns}}{\phi} &= \frac{25.4 \text{ kip-in./ft}}{0.90} \\
&= 28.2 \text{ kip-in./ft}
\end{aligned}$$

The area of steel provided is,

$$A_s = 0.150 \text{ in.}^2/\text{ft}$$

The reduced slab flexural strength is,

$$\begin{aligned}
M_{nsr} &= \frac{l_h}{l_{dh}}\left(1 - \frac{N_{uw}\ /\ \phi f_y}{A_s}\right) M_{ns} \\
&= \frac{9.75 \text{ in.}}{10 \text{ in.}}\left[1 - \frac{(0.280 \text{ kip/ft})\ /\left[0.90\left(60 \text{ ksi}\right)\right]}{0.150 \text{ in.}^2}\right] 28.2 \text{ kip-in./ft} \\
&= 26.5 \text{ kip-in./ft}
\end{aligned}$$

$$\begin{aligned}
\phi M_{nsr} &= 0.90(26.5 \text{ kip-in./ft}) \\
&= 23.9 \text{ kip-in./ft} > 11.3 \text{ kip-in./ft} \quad \textbf{o.k.}
\end{aligned}$$

The area of steel required for direct tension is sufficiently small that either method provides an acceptable result.

The type of cladding support used in this example presents several difficulties for construction tolerances, and for that reason, it is not preferred over other means of cladding support. Although there may be vertical slotted holes in the vertical leg of the shelf angle, there is substantially less room for vertical adjustment of the shelf angle compared with the structures considered in previous examples. Furthermore, the only means to adjust the angle toward or away from the slab edge is to adjust the bent plate to spandrel beam connection since the vertical leg of the angle is welded directly to the pour stop. However, there may be situations in which this type of cladding support is the best solution, particularly where there is no room for kickers beneath the slab or the shelf angle height is relatively high for architectural reasons.

Table 7-1. Vertical Deflection at Tip of Shelf Angle, in. Supporting 10 Vertical Feet of Brick[1]										
Angle	Thickness, in.	Spacing of Angle Attachment to Structure, in.								
		24	30	36	42	48	54	60	66	72
L5x5	5/16	0.0258	0.0329	0.0399	0.0467	0.0534	0.0651	0.0788	0.0949	0.114
	3/8	0.0151	0.0193	0.0234	0.0274	0.0316	0.0386	0.0468	0.0565	0.0679
	7/16[(8)]	0.00965	0.0123	0.0149	0.0175	0.0204	0.0250	0.0304	0.0368	0.0444
	1/2	0.00653	0.00834	0.0101	0.0119	0.0140	0.0171	0.0209	0.0254	0.0307
	5/8	0.00340	0.00435	0.00527	0.00618	0.00743	0.00917	0.0113	0.0138	0.0168
	3/4	0.00200	0.00256	0.00311	0.00365	0.00448	0.00557	0.00689	0.00851	0.0105
L6x4 (LLH)	5/16	0.0491	0.0624	0.0755	0.0883	0.1008	0.117	0.142	0.171	–
	3/8	0.0286	0.0364	0.0441	0.0516	0.0589	0.0694	0.0842	0.102	0.122
	7/16[(8)]	0.0182	0.0232	0.0281	0.0328	0.0375	0.0447	0.0544	0.0659	0.0793
	1/2	0.0123	0.0156	0.0189	0.0222	0.0253	0.0305	0.0373	0.0452	0.0547
	9/16[(8)]	0.00870	0.0111	0.0134	0.0157	0.0179	0.0219	0.0268	0.0327	0.0397
	5/8	0.00639	0.00813	0.00985	0.0115	0.0132	0.0163	0.0200	0.0245	0.0299
	3/4	0.00374	0.00477	0.00578	0.00676	0.00793	0.00985	0.0122	0.0151	0.0186
L6x6	5/16	0.0420	0.0538	0.0655	0.0769	0.0881	0.0995	0.119	0.141	0.167
	3/8	0.0246	0.0316	0.0384	0.0451	0.0517	0.0590	0.0705	0.0837	0.0989
	7/16[(8)]	0.0156	0.0201	0.0245	0.0288	0.0330	0.0379	0.0453	0.0539	0.0638
	1/2	0.0106	0.0136	0.0165	0.0194	0.0223	0.0258	0.0308	0.0367	0.0435
	9/16[(8)]	0.00748	0.00961	0.0117	0.0138	0.0158	0.0184	0.0220	0.0263	0.0312
	5/8	0.00549	0.00705	0.00859	0.0101	0.0116	0.0136	0.0163	0.0195	0.0232
	3/4	0.00322	0.00414	0.00505	0.00594	0.00681	0.00811	0.00977	0.0117	0.0140
L7x4 (LLH)	3/8	0.0433	0.0552	0.0668	0.0781	0.0893	0.100	0.118	0.141	0.168
	1/2	0.0185	0.0236	0.0286	0.0335	0.0383	0.0430	0.0519	0.0624	0.0746
	5/8	0.00963	0.0123	0.0149	0.0174	0.0199	0.0228	0.0277	0.0335	0.0403
	3/4	0.00564	0.00719	0.00872	0.0102	0.0117	0.0137	0.0167	0.0203	0.0246
L8x4 (LLH)	1/2	0.0261	0.0334	0.0405	0.0474	0.0542	0.0608	0.0689	0.0822	0.0975
	9/16[(8)]	0.0185	0.0236	0.0286	0.0336	0.0384	0.0430	0.0493	0.0589	0.0701
	3/4	0.00795	0.0102	0.0123	0.0145	0.0165	0.0186	0.0220	0.0265	0.0318
L8x6 (LLH)	7/16[(8)]	0.0345	0.0444	0.0542	0.0638	0.0732	0.0823	0.0913	0.107	0.125
	1/2	0.0232	0.0300	0.0366	0.0430	0.0494	0.0556	0.0617	0.0726	0.0850
	9/16[(8)]	0.0164	0.0212	0.0258	0.0304	0.0349	0.0393	0.0439	0.0516	0.0605
	5/8	0.0120	0.0155	0.0189	0.0223	0.0256	0.0288	0.0323	0.0380	0.0446
	3/4	0.00702	0.00906	0.0111	0.0130	0.0150	0.0168	0.0191	0.0226	0.0265
L8x6 (LLV)	7/16[(8)]	0.0138	0.0179	0.0219	0.0258	0.0297	0.0338	0.0399	0.0470	0.0550
	1/2	0.00928	0.0120	0.0148	0.0174	0.0201	0.0229	0.0271	0.0319	0.0374
	9/16[(8)]	0.00656	0.00852	0.0104	0.0123	0.0142	0.0163	0.0193	0.0227	0.0266
	5/8	0.00480	0.00624	0.00765	0.00903	0.0104	0.0120	0.0142	0.0167	0.0197
	3/4	0.00281	0.00365	0.00448	0.00529	0.00609	0.00708	0.00841	0.00994	0.01170
L8x8	1/2	0.0205	0.0267	0.0328	0.0388	0.0447	0.0504	0.0560	0.0644	0.0748
	9/16[(8)]	0.0145	0.0189	0.0232	0.0275	0.0316	0.0357	0.0397	0.0458	0.0532
	5/8	0.0107	0.0139	0.0171	0.0202	0.0232	0.0262	0.0292	0.0339	0.0394
	3/4	0.00622	0.00811	0.0100	0.0118	0.0136	0.0153	0.0171	0.0200	0.0232

Notes:
1. Brick is 40 psf, standard clay masonry (3⅝ in. thickness).
2. Table is based on methodology in Tide and Krogstad (1993).
3. Angle material is ASTM A36 steel.
4. Calculations assume that at least two-thirds of the width of the brick veneer bears on horizontal leg of the shelf angle and that the brick load is transmitted to shelf angle ½-in. outboard of the interior face of brick (Tide and Krogstad, 1993).
5. Shims locations are assumed at the anchor rods with height equal to that of the vertical leg of the angle and width of 3 in.
6. Angle gage is 2½ in. for 4-in. leg, 3 in. for 5-in. leg, 3½ in. for 6-in. leg, 4 in. for 7-in. leg, and 4½ in. for 8 in. leg.
7. Coefficient of friction of 0.2 is used to account for frictional effects between the brick and the shelf angle.
8. Note that 7⁄16-in. and 9⁄16-in.-thick angles may be difficult to obtain.
9. Where no value is given, the angle is overstressed.

Table 7-2. Vertical Deflection at Tip of Shelf Angle, in.
Supporting 12 Vertical Feet of Brick[1]

Angle	Thickness, in.	Spacing of Angle Attachment to Structure, in.								
		24	30	36	42	48	54	60	66	72
L5x5	5/16	0.0311	0.0397	0.0482	0.0566	0.0648	0.0739	0.0887	0.106	0.125
	3/8	0.0182	0.0233	0.0283	0.0332	0.0380	0.0438	0.0526	0.0629	0.0748
	7/16[8]	0.0116	0.0149	0.0181	0.0212	0.0243	0.0284	0.0342	0.0409	0.0488
	1/2	0.00787	0.0101	0.0122	0.0143	0.0164	0.0194	0.0235	0.0282	0.0337
	5/8	0.00410	0.00524	0.00637	0.00748	0.00857	0.0104	0.0126	0.0153	0.0184
	3/4	0.00241	0.00309	0.00376	0.00441	0.00512	0.00629	0.00769	0.00939	0.0114
L6x4 (LLH)	5/16	0.0591	0.0753	0.0912	0.107	0.122	0.137	-	-	-
	3/8	0.0345	0.0440	0.0533	0.0624	0.0714	0.0803	0.0946	0.113	0.134
	7/16[8]	0.0219	0.0280	0.0339	0.0398	0.0455	0.0511	0.0611	0.0732	0.0873
	1/2	0.0148	0.0189	0.0229	0.0268	0.0307	0.0346	0.0418	0.0502	0.0601
	9/16[8]	0.0105	0.0134	0.0162	0.0190	0.0217	0.0248	0.0300	0.0362	0.0435
	5/8	0.00769	0.00981	0.0119	0.0140	0.0160	0.0184	0.0224	0.0271	0.0328
	3/4	0.00451	0.00576	0.00698	0.00819	0.00938	0.0111	0.0136	0.0166	0.0203
L6x6	5/16	0.0505	0.0649	0.0791	0.0931	0.107	0.120	0.135	0.159	0.186
	3/8	0.0296	0.0381	0.0464	0.0546	0.0627	0.0706	0.0801	0.0943	0.110
	7/16[8]	0.0188	0.0242	0.0296	0.0348	0.0399	0.0450	0.0515	0.0607	0.0712
	1/2	0.0127	0.0164	0.0200	0.0235	0.0270	0.0304	0.0350	0.0413	0.0486
	9/16[8]	0.00900	0.0116	0.0141	0.0166	0.0191	0.0215	0.0250	0.0296	0.0348
	5/8	0.00661	0.00850	0.0104	0.0122	0.0140	0.0158	0.0185	0.0219	0.0258
	3/4	0.00388	0.00500	0.00610	0.00718	0.00825	0.00930	0.0111	0.0132	0.0156
L7x4 (LLH)	3/8	0.0521	0.0665	0.0807	0.0946	0.108	0.122	0.135	0.158	-
	1/2	0.0223	0.0285	0.0346	0.0406	0.0465	0.0522	0.0585	0.0697	0.0826
	5/8	0.0116	0.0148	0.0180	0.0211	0.0242	0.0272	0.0312	0.0373	0.0445
	3/4	0.00679	0.00868	0.0105	0.0124	0.0142	0.0159	0.0187	0.0226	0.0271
L8x4 (LLH)	1/2	0.0315	0.0403	0.0489	0.0574	0.0657	0.0739	0.0819	0.0923	0.109
	9/16[8]	0.0222	0.0285	0.0346	0.0406	0.0465	0.0523	0.0580	0.0661	0.0780
	3/4	0.0096	0.0123	0.0149	0.0175	0.0200	0.0225	0.0250	0.0296	0.0352
L8x6 (LLH)	7/16[8]	0.0415	0.0536	0.0655	0.0772	0.0887	0.100	0.111	0.122	0.141
	1/2	0.0280	0.0361	0.0442	0.0521	0.0598	0.0675	0.0750	0.0824	0.0957
	9/16[8]	0.0197	0.0255	0.0312	0.0368	0.0423	0.0477	0.0530	0.0586	0.0681
	5/8	0.0145	0.0187	0.0228	0.0269	0.0310	0.0349	0.0388	0.0432	0.0502
	3/4	0.00845	0.0109	0.0134	0.0158	0.0181	0.0204	0.0227	0.0256	0.0298
L8x6 (LLV)	7/16[8]	0.0166	0.0215	0.0264	0.0312	0.0360	0.0406	0.0458	0.0534	0.0620
	1/2	0.0112	0.0145	0.0178	0.0211	0.0243	0.0275	0.0311	0.0363	0.0422
	9/16[8]	0.00790	0.0103	0.0126	0.0149	0.0172	0.0194	0.0221	0.0258	0.0300
	5/8	0.00578	0.00752	0.00923	0.0109	0.0126	0.0142	0.0163	0.0190	0.0221
	3/4	0.00338	0.00440	0.00541	0.00640	0.00737	0.00834	0.00963	0.0113	0.0132
L8x8	1/2	0.0247	0.0322	0.0396	0.0469	0.0541	0.0611	0.0681	0.0749	0.0849
	9/16[8]	0.0175	0.0228	0.0280	0.0332	0.0383	0.0433	0.0482	0.0530	0.0604
	5/8	0.0128	0.0167	0.0206	0.0244	0.0281	0.0318	0.0354	0.0390	0.0447
	3/4	0.00748	0.00978	0.0120	0.0143	0.0164	0.0186	0.0207	0.0229	0.0264

Notes:
1. Brick is 40 psf, standard clay masonry (3⅝ in. thickness).
2. Table is based on methodology in Tide and Krogstad (1993).
3. Angle material is ASTM A36 steel.
4. Calculations assume that at least two-thirds of the width of the brick veneer bears on horizontal leg of the shelf angle and that the brick load is transmitted to shelf angle ½-in. outboard of the interior face of brick (Tide and Krogstad, 1993).
5. Shims locations are assumed at the anchor rods with height equal to that of the vertical leg of the angle and width of 3 in.
6. Angle gage is 2½ in. for 4-in. leg, 3 in. for 5-in. leg, 3½ in. for 6-in. leg, 4 in. for 7-in. leg, and 4½ in. for 8 in. leg.
7. Coefficient of friction of 0.2 is used to account for frictional effects between the brick and the shelf angle.
8. Note that 7⁄16-in. and 9⁄16-in.-thick angles may be difficult to obtain.
9. Where no value is given, the angle is overstressed.

Table 7-3. Vertical Deflection at Tip of Shelf Angle, in. Supporting 14 Vertical Feet of Brick[1]										
Angle	**Thickness, in.**	**Spacing of Angle Attachment to Structure, in.**								
		24	**30**	**36**	**42**	**48**	**54**	**60**	**66**	**72**
L5x5	**5/16**	0.0364	0.0465	0.0565	0.0664	0.0762	0.0858	0.0985	0.117	-
	3/8	0.0213	0.0273	0.0331	0.0389	0.0446	0.0503	0.0584	0.0693	0.0818
	7/16[8]	0.0136	0.0174	0.0212	0.0249	0.0285	0.0321	0.0379	0.0450	0.0533
	1/2	0.00921	0.0118	0.0143	0.0168	0.0193	0.0218	0.0260	0.0310	0.0368
	5/8	0.00480	0.00614	0.00747	0.00878	0.0101	0.0116	0.0139	0.0167	0.0200
	3/4	0.00282	0.00362	0.00440	0.00518	0.00594	0.00701	0.00849	0.0103	0.0124
L6x4 (LLH)	**5/16**	0.0691	0.0882	0.107	0.126	0.144	-	-	-	-
	3/8	0.0404	0.0515	0.0625	0.0733	0.0840	0.0945	0.1050	0.124	-
	7/16[8]	0.0257	0.0328	0.0398	0.0467	0.0535	0.0602	0.0677	0.0805	0.0953
	1/2	0.0173	0.0221	0.0268	0.0315	0.0361	0.0406	0.0463	0.0552	0.0656
	9/16[8]	0.0123	0.0157	0.0190	0.0223	0.0256	0.0288	0.0332	0.0398	0.0474
	5/8	0.00900	0.0115	0.0140	0.0164	0.0188	0.0211	0.0248	0.0297	0.0356
	3/4	0.00528	0.00674	0.00819	0.00961	0.0110	0.0124	0.0150	0.0182	0.0219
L6x6	**5/16**	0.0591	0.0760	0.0928	0.109	0.126	0.142	0.157	-	-
	3/8	0.0346	0.0446	0.0544	0.0641	0.0737	0.0831	0.0924	0.1049	0.122
	7/16[8]	0.0220	0.0284	0.0346	0.0408	0.0469	0.0529	0.0589	0.0675	0.0786
	1/2	0.0149	0.0192	0.0234	0.0276	0.0317	0.0358	0.0398	0.0459	0.0536
	9/16[8]	0.0105	0.0136	0.0166	0.0195	0.0224	0.0253	0.0282	0.0328	0.0384
	5/8	0.00773	0.0100	0.0122	0.0143	0.0165	0.0186	0.0207	0.0243	0.0285
	3/4	0.00454	0.00585	0.00715	0.00843	0.00969	0.0109	0.0124	0.0146	0.0171
L7x4 (LLH)	**3/8**	0.0610	0.0779	0.0946	0.111	0.127	0.143	-	-	-
	1/2	0.0261	0.0334	0.0406	0.0477	0.0546	0.0615	0.0682	0.0770	0.0906
	5/8	0.0136	0.0174	0.0211	0.0248	0.0284	0.0320	0.0355	0.0412	0.0487
	3/4	0.00794	0.0102	0.0124	0.0145	0.0166	0.0187	0.0208	0.0248	0.0296
L8x4 (LLH)	**1/2**	0.0368	0.0471	0.0573	0.0674	0.0773	0.0870	0.0966	0.106	0.120
	9/16[8]	0.0260	0.0333	0.0406	0.0477	0.0547	0.0616	0.0684	0.0750	0.0858
	3/4	0.0112	0.0143	0.0175	0.0205	0.0236	0.0265	0.0295	0.0328	0.0386
L8x6 (LLH)	**7/16[8]**	0.0485	0.0627	0.0767	0.0905	0.104	0.118	0.131	0.144	0.157
	1/2	0.0327	0.0423	0.0517	0.0611	0.0703	0.0793	0.0883	0.0971	0.106
	9/16[8]	0.0231	0.0299	0.0366	0.0432	0.0497	0.0561	0.0624	0.0686	0.0757
	5/8	0.0169	0.0219	0.0268	0.0316	0.0364	0.0411	0.0457	0.0503	0.0558
	3/4	0.00987	0.0128	0.0157	0.0185	0.0213	0.0240	0.0268	0.0294	0.0331
L8x6 (LLV)	**7/16[8]**	0.0194	0.0252	0.0309	0.0366	0.0422	0.0478	0.0533	0.0598	0.0690
	1/2	0.0131	0.0170	0.0209	0.0247	0.0285	0.0323	0.0360	0.0406	0.0469
	9/16[8]	0.00923	0.0120	0.0148	0.0175	0.0202	0.0228	0.0254	0.0289	0.0334
	5/8	0.00676	0.00880	0.0108	0.0128	0.0148	0.0167	0.0186	0.0213	0.0246
	3/4	0.00396	0.00515	0.00634	0.00750	0.00866	0.00980	0.0109	0.0126	0.0146
L8x8	**1/2**	0.0289	0.0377	0.0464	0.0550	0.0635	0.0719	0.0801	0.0883	0.0963
	9/16[8]	0.0204	0.0266	0.0328	0.0389	0.0449	0.0509	0.0567	0.0625	0.0681
	5/8	0.0150	0.0196	0.0241	0.0286	0.0330	0.0374	0.0417	0.0459	0.0501
	3/4	0.00875	0.0114	0.0141	0.0167	0.0193	0.0219	0.0244	0.0269	0.0295

Notes:
1. Brick is 40 psf, standard clay masonry (3⅝ in. thickness).
2. Table is based on methodology in Tide and Krogstad (1993).
3. Angle material is ASTM A36 steel.
4. Calculations assume that at least two-thirds of the width of the brick veneer bears on horizontal leg of the shelf angle and that the brick load is transmitted to shelf angle ½-in. outboard of the interior face of brick (Tide and Krogstad, 1993).
5. Shims locations are assumed at the anchor rods with height equal to that of the vertical leg of the angle and width of 3 in.
6. Angle gage is 2½ in. for 4-in. leg, 3 in. for 5-in. leg, 3½ in. for 6-in. leg, 4 in. for 7-in. leg, and 4½ in. for 8 in. leg.
7. Coefficient of friction of 0.2 is used to account for frictional effects between the brick and the shelf angle.
8. Note that 7/16-in. and 9/16-in.-thick angles may be difficult to obtain.
9. Where no value is given, the angle is overstressed.

Table 7-4. Vertical Deflection at Tip of Shelf Angle, in.
Supporting 16 Vertical Feet of Brick[1]

Angle	Thickness, in.	Spacing of Angle Attachment to Structure, in.								
		24	30	36	42	48	54	60	66	72
L5x5	5/16	0.0417	0.0533	0.0649	0.0763	0.0875	0.0986	0.110	-	-
	3/8	0.0244	0.0312	0.0380	0.0447	0.0513	0.0578	0.0643	0.0757	0.0888
	7/16[8]	0.0156	0.0200	0.0243	0.0286	0.0328	0.0370	0.0416	0.0492	0.0578
	1/2	0.0105	0.0135	0.0164	0.0193	0.0222	0.0250	0.0285	0.0338	0.0399
	5/8	0.00549	0.00704	0.00857	0.0101	0.0116	0.0131	0.0153	0.0182	0.0216
	3/4	0.00324	0.00415	0.00505	0.00594	0.00683	0.00773	0.00929	0.0111	0.0134
L6x4 (LLH)	5/16	0.0792	0.101	0.123	0.144	-	-	-	-	-
	3/8	0.0462	0.0590	0.0717	0.0842	0.0965	0.109	-	-	-
	7/16[8]	0.0294	0.0376	0.0456	0.0536	0.0615	0.0692	0.0769	0.0879	0.103
	1/2	0.0198	0.0254	0.0308	0.0362	0.0415	0.0467	0.0519	0.0602	0.0710
	9/16[8]	0.0140	0.0179	0.0218	0.0256	0.0294	0.0331	0.0368	0.0433	0.0513
	5/8	0.0103	0.0132	0.0160	0.0188	0.0216	0.0243	0.0272	0.0324	0.0384
	3/4	0.00604	0.00773	0.00939	0.0110	0.0127	0.0143	0.0164	0.0197	0.0236
L6x6	5/16	0.0676	0.0871	0.106	0.125	0.144	0.163	-	-	-
	3/8	0.0396	0.0511	0.0624	0.0736	0.0846	0.0956	0.106	0.117	0.134
	7/16[8]	0.0252	0.0325	0.0397	0.0469	0.0539	0.0609	0.0678	0.0745	0.0860
	1/2	0.0170	0.0220	0.0268	0.0316	0.0364	0.0411	0.0458	0.0506	0.0586
	9/16[8]	0.0121	0.0155	0.0190	0.0224	0.0258	0.0291	0.0324	0.0361	0.0419
	5/8	0.00884	0.0114	0.0139	0.0165	0.0189	0.0214	0.0238	0.0267	0.0311
	3/4	0.00519	0.00670	0.00820	0.00967	0.0111	0.0126	0.0140	0.0160	0.0187
L7x4 (LLH)	3/8	0.0698	0.0893	0.109	0.128	0.146	-	-	-	-
	1/2	0.0299	0.0383	0.0466	0.0547	0.0628	0.0708	0.0786	0.0863	0.0986
	5/8	0.0155	0.0199	0.0242	0.0285	0.0327	0.0368	0.0409	0.0450	0.0529
	3/4	0.00909	0.0116	0.0142	0.0167	0.0191	0.0216	0.0240	0.0271	0.0321
L8x4 (LLH)	1/2	0.0421	0.0540	0.0658	0.0774	0.0888	0.100	0.111	0.122	0.133
	9/16[8]	0.0298	0.0382	0.0465	0.0547	0.0629	0.0708	0.0787	0.0865	0.0941
	3/4	0.0128	0.0164	0.0200	0.0236	0.0271	0.0305	0.0339	0.0373	0.0421
L8x6 (LLH)	7/16[8]	0.0555	0.0718	0.0880	0.104	0.120	0.135	0.151	0.166	0.181
	1/2	0.0374	0.0484	0.0593	0.0701	0.0807	0.0912	0.102	0.112	0.122
	9/16[8]	0.0264	0.0342	0.0419	0.0495	0.0570	0.0645	0.0718	0.0790	0.0862
	5/8	0.0193	0.0251	0.0307	0.0363	0.0418	0.0472	0.0526	0.0579	0.0631
	3/4	0.0113	0.0146	0.0180	0.0212	0.0245	0.0276	0.0308	0.0339	0.0370
L8x6 (LLV)	7/16[8]	0.0222	0.0289	0.0355	0.0420	0.0485	0.0549	0.0613	0.0675	0.0761
	1/2	0.0150	0.0195	0.0240	0.0284	0.0328	0.0371	0.0414	0.0456	0.0517
	9/16[8]	0.0106	0.0138	0.0169	0.0201	0.0232	0.0262	0.0293	0.0323	0.0368
	5/8	0.00774	0.0101	0.0124	0.0147	0.0170	0.0192	0.0214	0.0236	0.0271
	3/4	0.00453	0.00590	0.00726	0.00861	0.00994	0.0113	0.0126	0.0140	0.0161
L8x8	1/2	0.0330	0.0432	0.0532	0.0631	0.0729	0.0826	0.0922	0.102	0.111
	9/16[8]	0.0233	0.0305	0.0376	0.0446	0.0516	0.0584	0.0652	0.0719	0.0785
	5/8	0.0171	0.0224	0.0277	0.0328	0.0379	0.0430	0.0480	0.0529	0.0578
	3/4	0.0100	0.0131	0.0162	0.0192	0.0222	0.0251	0.0280	0.0309	0.0338

Notes:
1. Brick is 40 psf, standard clay masonry (3⅝ in. thickness).
2. Table is based on methodology in Tide and Krogstad (1993).
3. Angle material is ASTM A36 steel.
4. Calculations assume that at least two-thirds of the width of the brick veneer bears on horizontal leg of the shelf angle and that the brick load is transmitted to shelf angle ½-in. outboard of the interior face of brick (Tide and Krogstad, 1993).
5. Shims locations are assumed at the anchor rods with height equal to that of the vertical leg of the angle and width of 3 in.
6. Angle gage is 2½ in. for 4-in. leg, 3 in. for 5-in. leg, 3½ in. for 6-in. leg, 4 in. for 7-in. leg, and 4½ in. for 8 in. leg.
7. Coefficient of friction of 0.2 is used to account for frictional effects between the brick and the shelf angle.
8. Note that 7/16-in. and 9/16-in.-thick angles may be difficult to obtain.
9. Where no value is given, the angle is overstressed.

Table 7-5. Vertical Deflection at Tip of Shelf Angle, in. Supporting 18 Vertical Feet of Brick[1]										
Angle	**Thickness, in.**	**Spacing of Angle Attachment to Structure, in.**								
		24	**30**	**36**	**42**	**48**	**54**	**60**	**66**	**72**
L5x5	**5/16**	0.0470	0.0601	0.0732	0.0861	0.0989	0.112	-	-	-
	3/8	0.0275	0.0352	0.0429	0.0504	0.0579	0.0654	0.0727	0.0821	0.0957
	7/16(8)	0.0176	0.0225	0.0274	0.0322	0.0370	0.0418	0.0465	0.0533	0.0623
	1/2	0.0119	0.0152	0.0185	0.0218	0.0251	0.0283	0.0315	0.0366	0.0429
	5/8	0.0062	0.00794	0.00967	0.0114	0.0131	0.0148	0.0166	0.0197	0.0232
	3/4	0.0036	0.00468	0.00570	0.00671	0.00771	0.00870	0.0101	0.0120	0.0143
L6x4 (LLH)	**5/16**	0.0892	0.114	0.139	-	-	-	-	-	-
	3/8	0.0521	0.0666	0.0809	0.0951	0.109	-	-	-	-
	7/16(8)	0.0331	0.0424	0.0515	0.0605	0.0695	0.0783	0.0870	0.0957	-
	1/2	0.0224	0.0286	0.0348	0.0409	0.0469	0.0529	0.0588	0.0651	0.0764
	9/16(8)	0.0158	0.0202	0.0246	0.0289	0.0332	0.0374	0.0416	0.0468	0.0551
	5/8	0.0116	0.0149	0.0181	0.0212	0.0244	0.0275	0.0306	0.0350	0.0413
	3/4	0.00681	0.00871	0.0106	0.0125	0.0143	0.0161	0.0179	0.0213	0.0253
L6x6	**5/16**	0.0762	0.0982	0.120	0.142	0.163	-	-	-	-
	3/8	0.0447	0.0576	0.0704	0.0831	0.0956	0.1080	0.1203	0.132	-
	7/16(8)	0.0284	0.0367	0.0448	0.0529	0.0609	0.0688	0.0766	0.0844	0.0934
	1/2	0.0192	0.0248	0.0303	0.0357	0.0411	0.0465	0.0518	0.0570	0.0637
	9/16(8)	0.0136	0.0175	0.0214	0.0253	0.0291	0.0329	0.0367	0.0404	0.0455
	5/8	0.0100	0.0129	0.0157	0.0186	0.0214	0.0242	0.0269	0.0297	0.0337
	3/4	0.00585	0.00756	0.00925	0.0109	0.0126	0.0142	0.0158	0.0175	0.0203
L7x4 (LLH)	**3/8**	0.0786	0.101	0.123	0.144	-	-	-	-	-
	1/2	0.0337	0.0432	0.0525	0.0618	0.0710	0.0800	0.0890	0.0978	0.107
	5/8	0.0175	0.0224	0.0273	0.0321	0.0369	0.0416	0.0463	0.0509	0.0571
	3/4	0.0102	0.0131	0.0160	0.0188	0.0216	0.0244	0.0271	0.0298	0.0345
L8x4 (LLH)	**1/2**	0.0474	0.0609	0.0742	0.0874	0.1004	0.113	0.126	0.138	-
	9/16(8)	0.0335	0.0431	0.0525	0.0618	0.0710	0.0801	0.0891	0.0980	0.107
	3/4	0.0144	0.0185	0.0226	0.0266	0.0306	0.0345	0.0384	0.0422	0.0460
L8x6 (LLH)	**7/16(8)**	0.0625	0.0809	0.0992	0.117	0.135	0.153	0.170	0.188	-
	1/2	0.0421	0.0546	0.0669	0.0791	0.0912	0.103	0.115	0.127	0.138
	9/16(8)	0.0297	0.0386	0.0473	0.0559	0.0644	0.0729	0.0812	0.0895	0.0976
	5/8	0.0218	0.0282	0.0346	0.0409	0.0472	0.0534	0.0595	0.0655	0.0715
	3/4	0.0127	0.0165	0.0203	0.0240	0.0276	0.0312	0.0348	0.0384	0.0419
L8x6 (LLV)	**7/16(8)**	0.0250	0.0325	0.0400	0.0474	0.0548	0.0621	0.0693	0.0764	0.0835
	1/2	0.0168	0.0220	0.0270	0.0320	0.0370	0.0419	0.0468	0.0516	0.0564
	9/16(8)	0.0119	0.0155	0.0191	0.0226	0.0262	0.0296	0.0331	0.0365	0.0401
	5/8	0.00870	0.0114	0.0140	0.0166	0.0192	0.0217	0.0242	0.0267	0.0296
	3/4	0.00510	0.00665	0.00819	0.00972	0.0112	0.0127	0.0142	0.0157	0.0175
L8x8	**1/2**	0.0372	0.0486	0.0600	0.0712	0.0823	0.0933	0.104	0.115	0.126
	9/16(8)	0.0263	0.0344	0.0424	0.0504	0.0582	0.0660	0.0737	0.0814	0.0889
	5/8	0.0193	0.0253	0.0312	0.0370	0.0428	0.0486	0.0542	0.0598	0.0654
	3/4	0.0113	0.0148	0.0182	0.0216	0.0250	0.0284	0.0317	0.0350	0.0382

Notes:
1. Brick is 40 psf, standard clay masonry (3⅝ in. thickness).
2. Table is based on methodology in Tide and Krogstad (1993).
3. Angle material is ASTM A36 steel.
4. Calculations assume that at least two-thirds of the width of the brick veneer bears on horizontal leg of the shelf angle and that the brick load is transmitted to shelf angle ½-in. outboard of the interior face of brick (Tide and Krogstad, 1993).
5. Shims locations are assumed at the anchor rods with height equal to that of the vertical leg of the angle and width of 3 in.
6. Angle gage is 2½ in. for 4-in. leg, 3 in. for 5-in. leg, 3½ in. for 6-in. leg, 4 in. for 7-in. leg, and 4½ in. for 8 in. leg.
7. Coefficient of friction of 0.2 is used to account for frictional effects between the brick and the shelf angle.
8. Note that 7/16-in. and 9/16-in.-thick angles may be difficult to obtain.
9 Where no value is given, the angle is overstressed.

Chapter 8
PRECAST CONCRETE WALL PANELS

8.1 GENERAL DESCRIPTION OF PRECAST CONCRETE WALL PANEL SYSTEMS

Precast concrete wall panels have many applications, including those where the concrete panel is a structural load bearing element. This design guide, however, addresses only those applications where the concrete wall panels are used as a curtain wall supported by a steel frame, where each panel is supported from the structure and does not interact with the primary building structure or become an unintended load path for either gravity or lateral loads. In most applications, panels may be removed individually without affecting the support of adjacent panels. Predominantly, panels used in curtain wall applications are limited in height to the floor-to-floor dimension (although the joints between panels may not coincide with the elevations of the floors) and limited in width to the width of a structural bay. Some panels are not as large as a full story height and a full bay width, but rather may be vertical strips, horizontal strips, panels with punched window openings, or spandrels, mullions, or column covers.

Another application addressed in this design guide involves panels supported on the foundation wall with the steel building structure providing only lateral support for out-of-plane loads. Such wall panel units may be stacked so that lower wall panels support the weight of upper panels. The larger dimension of these panels tends to be vertical and at least one story high.

Precast concrete panels are either solid concrete panels or sandwich panels, which have a layer of rigid insulation "sandwiched" between an exterior layer of architectural concrete and an interior layer of concrete that serves as the structural back-up. The sandwich panel is manufactured so that the two layers of concrete are tied together and the exterior layer is supported both vertically and laterally by the back-up panel. The back-up panel is supported by the primary building structure.

Both panel types function as barrier walls, and cracking must be limited to hairline widths to prevent moisture penetration through the panel. The joints between panels are the key to successful moisture protection performance of the wall system. The system uses sealants to make the joints watertight, and movement at these joints must be considered when designing the support and attachment of the panels. The life expectancy of the sealant joint depends on, among other factors, the number of movement cycles and the magnitude of the movement as a percentage of joint width.

When solid wall panels are used, insulation is affixed to the interior surface, or an independent insulated partition wall is placed inside the concrete panel.

8.2 STRATEGIES FOR SUPPORT OF PRECAST CONCRETE WALL PANELS

The most important strategy for support of precast concrete panels is to support the weight of each panel by no more than two load bearing connections. This keeps the support for gravity loads determinate so that calculation of the reactions is independent of the support flexibility. A panel should not be supported at three points because there is no practical way to erect the panels and ensure that the load at each bearing connection (one at each end and one in the middle) is as assumed in design. For example, if the panel is initially set on shims at one of the ends and in the middle, the middle support will receive more than the 62.5 percent of the load assumed for the center support of the two-span continuous condition. When the third support is eventually shimmed, the load sharing of the supports will be changed. Shimming to a snug-tight condition will not change the initial distribution significantly. Over-shimming may lift the support such that the center support is unloaded and each end will receive 50 percent of the weight. Even if careful shimming with load monitoring could be used to distribute the weight of the panel as assumed in the analysis, movement of any support location, such as that caused by deflection of a beam due to floor live loads, would once again change the support reactions. Besides making it difficult—if not impossible—to know the bearing reactions for design of the connections, more than two supports makes it impractical to know the final force distribution in the panel for proper design of the panel itself.

Another important strategy for the support of precast panels is accommodating volume change of the panels. Volume change will occur due to changes in temperature and, to a lesser degree, moisture. Drying shrinkage of the concrete panel will also have an impact, depending on its mix design and when it is erected relative to its production. To accommodate volume change, one of the bearing supports should allow lateral movement parallel to the width of the panel. This is commonly done with inserts or attachment clips. The other bearing connection transfers all the lateral forces parallel to the panel width into the support structure. All connections to the primary structure other than the two bearing

connections are tie-back connections only for out-of-plane support; they allow movement in any direction parallel to the plane of the panel.

As discussed in Chapter 1, structural integrity involves sufficient strength, ductility, and redundancy. Because redundancy is avoided in precast panel gravity attachments, it is important to compensate with ample strength and ductility in the attachments.

Figures 8-1 and 8-2 illustrate strategies for supporting wall panels that incorporate the above concepts. It is preferable to limit the number of panel-to-structure connections to four. Two bearing connections are provided at the top or bottom near the corners, with one of them allowing horizontal movement. Two tie-back connections are then provided near the other corners. Designers sometimes use additional tie-back connections if the panels are long, such as is shown in Figure 8-2b. However, the design of the panel and tie-back connections must consider the effects of panel bowing and the restraint provided by the redundant tie-back connections.

8.3 PARAMETERS AFFECTING THE DESIGN OF PRECAST CONCRETE WALL PANEL SUPPORTS

Parameters affecting the design of precast concrete panel supports include:

- Architectural layout of panels.
- Movement requirements.
- Magnitude of seismic forces.
- Field adjustability for tolerances and clearances.
- Durability.
- Fire-safing.

8.3.1 Architectural Layout of Panels

The architectural layout of precast panels is often based on a combination of considerations, including aesthetics, shipping and erection procedures, and economics. Panels can be arranged in a variety of patterns, as demonstrated in Figure 8-3. Panel cross sections in particular are often chosen for architectural reasons.

Aesthetics—Various types of wall panel systems include solid wall panels, spandrel panels, and "punched window" panels.

Shipping and Erection Procedures—Shipping and erection equipment capabilities often influence precast panel size, and must be considered in the selection of panel layout.

Economics—Panel sizes and shapes may be selected to minimize handling and erection time, which can be costly.

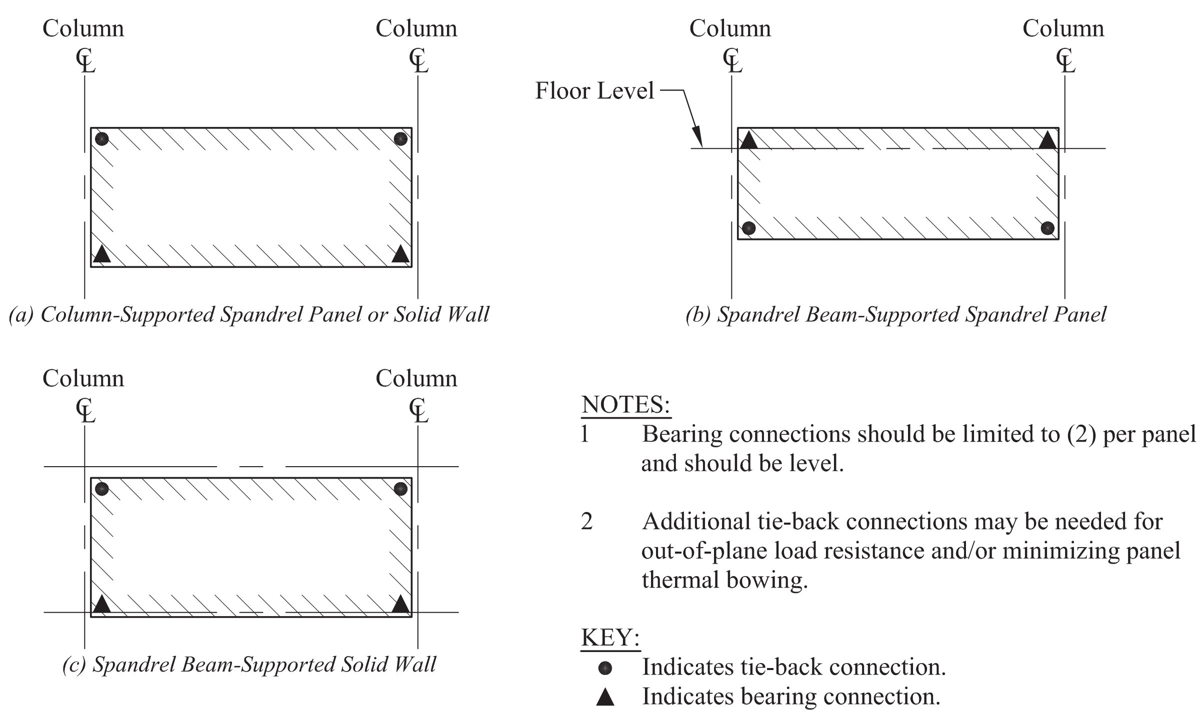
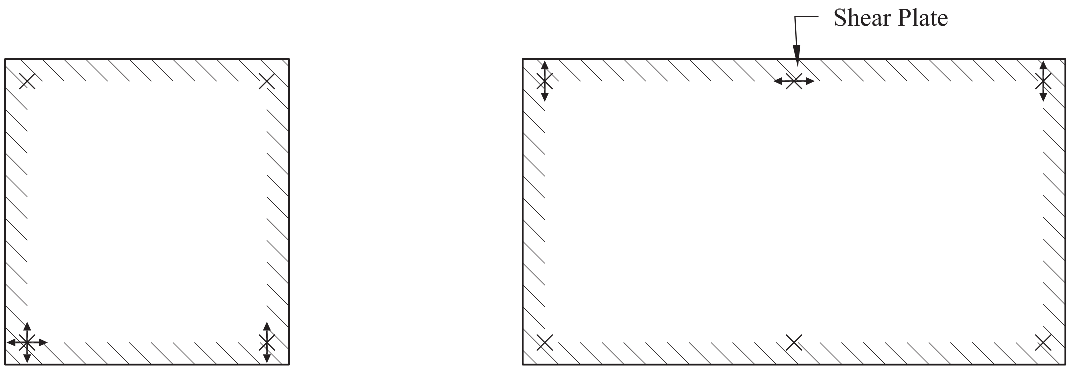


Fig. 8-1. Typical precast panel bearing and tie-back connections layout.



(a) Wall Panel Load Supports in Regions of Low Seismicity

(b) Spandrel Panel Load Supports Subject to Thermal Bowing and/or in Regions of High Seismicity

KEY:
Indicates direction of in-plane load resistance.
Indicates out-of-plane load resistance.

Fig. 8-2. Typical precast panel load supports.

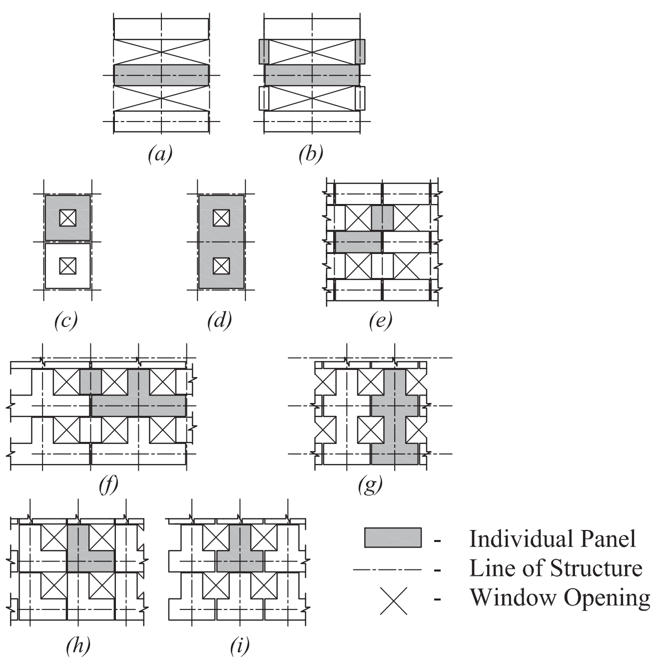


Fig. 8-3. Typical arrangement of precast concrete panels.
(Taken from reference "Architectural Precast Concrete," Second Edition, PCI. Used with permission Precast/Prestressed Concrete Institute.)

An economically efficient panel can be tilted immediately after its forms are stripped, and stored in the yard in a position similar to its final erected and adjusted position on the structure. Panels having similar connections increase overall efficiency. The workers' familiarity with the connections also results in increased productivity.

Panel layouts that are repetitive in finish, shape, and size will often also prove more economical.

8.3.2 Movement Requirements

Vertical and lateral movements, in-plane drift of the steel frame, and out-of-plane movements all require consideration.

Vertical and Lateral Movements—The supporting steel and attachments must accommodate the vertical and lateral movements of the precast panels due to volume change, which is caused by variations in temperature, creep, and shrinkage. Drying shrinkage and creep in precast concrete members continue to occur after erection and should be accounted for in connection and clearance detailing. Additionally, temperature differential between the interior of the building and the exterior may cause bowing of the precast panel.

- Design movement at horizontal soft joints: Horizontal soft joints must accommodate the movement due to the relative deflections of upper and lower floor structures, as well as the movement due to thermal and moisture volume changes in the precast panels. The sum of the movement accommodated by the soft joint will be approximately 25 to 50 percent of the soft-joint size, depending on sealant materials and sealant bond, to account for sealant compressibility and extensibility. Where window frames are attached to precast panels, the effects of panel movement on the window should be considered.

- Stiffness of supporting structure: The designer must provide enough stiffness to mitigate cracking of the precast panel due to support deflection. There is no industry rule of thumb for deflection criterion as there is with masonry cavity wall systems because each precast panel system behaves differently. The structural engineer of record (SER) must consider the deflection specific to the particular precast system and panel layout. Excessive deflection of a supporting spandrel beam can lead to translation and rotation of individual precast panels. The spandrel stiffness will often be controlled by the acceptable horizontal soft-joint size.

In-Plane Drift of the Steel Frame—The support and attachment details must allow the structural steel frame of the primary structure to undergo its design drift without restraint from the precast panels. As the structure undergoes drift, the upper floor moves laterally relative to the lower floor. In regions of low seismicity, the common strategy for accommodating this movement is to anchor the panels to the primary structure in such a manner that in-plane forces are resisted at only one anchor per panel. All other connections are detailed with allowance for in-plane movement. This is demonstrated in Figure 8-2b through the use of a shear plate, but it could also be achieved at one of the bearing connections. In high-seismic applications, movements are larger, and special attention may be required to allow for these deformations without inducing unanticipated force transfer in the panel.

Out-of-Plane Movement—The interior-to-exterior temperature differential can cause out-of-plane thermal bowing, particularly as the thickness of precast concrete panels decreases. Seismic forces can also cause bowing. Additional attachments to the primary structure are sometimes used to control bowing on long spandrel panels, and panel designs must consider the effects of this restraint on panel cracking. The restraining forces will be computed by the specialty structural engineer (SSE) designing the attachments. Although the magnitudes of these forces are usually not significant for the design of the primary structure, large forces may require consideration.

8.3.3 Magnitude of Seismic Forces

Precast concrete panels are one of the heavier façade systems and, therefore, possess larger seismic forces than most other systems. Greater allowances must be made in connections for seismic movement, and greater loads are imposed on the primary building structure with precast panels. The effects of these loads are accentuated by their typical eccentric loading to the support structure.

A common method of transferring large in-plane seismic loads is through the use of a shear plate connection. Figure 8-4 illustrates a common seismic shear plate detail for high seismic forces.

8.3.4 Field Adjustability for Tolerances and Clearances

As demonstrated in Chapter 7 with masonry cavity wall systems, precast concrete panel systems also require field adjustments for fabrication and erection tolerances of the steel frame. The magnitude of the necessary adjustments depends on the number of stories and the span lengths as discussed in Chapter 4. Suggested adjustments include:

- Means to adjust the slab edge in or out relative to the spandrel beam.

- Means to adjust the location of the precast panels in or out relative to the slab edge.

- Means to adjust the location of the precast panels up or down relative to the slab edge.

- Means to adjust the location of the attachments both vertically and horizontally.

A common method of providing vertical adjustment is through the use of shim plates. Care must be taken in the use of shims, as improperly designed shims can hinder movement and create unintended load paths. Leveling bolts tend to be more costly, but may be used in lieu of shims to save time in erection. Figure 8-5 demonstrates the typical use of these methods. The use of erection bolts in slotted holes with field welding is a common mechanism for achieving the necessary horizontal field adjustments.

8.3.5 Durability

Because the typical precast concrete panel façade system does not include a waterproof membrane, it is important that consideration be given to the moisture protection of the panel attachments. Designers should consider protective coatings, such as hot-dip galvanizing or zinc-rich paint for these elements. However, designers do not typically specify a protective coating for the primary structure for moisture from the exterior.

8.3.6 Fire-Safing

Fire-safing is required between the slab edge and the back of the precast panels, when precast concrete wall panels are set so they run by the edge of slab. A clearance between the slab edge and panels is required to accommodate erection tolerances, and fire-safing must be used to fill this gap.

If all of the steel frame erection tolerances are accommodated in the adjustment of the slab edge, the clearance is required only to accommodate the erection tolerance of the precast panels. If only part of the steel frame erection tolerance is accommodated in the adjustment of the slab edge, the clearance must also accommodate the balance.

Designers should consider the cost benefit of providing more slab edge adjustment and less clearance to reduce the amount of required fire-safing.

8.4 DESIGN RESPONSIBILITIES FOR PRECAST CONCRETE WALL PANELS

As discussed in Chapter 3, it is important that the design team understands the design responsibilities for the support of precast concrete walls. Contracts can define the design responsibilities for each project, and one approach commonly found on projects with precast concrete wall panels is discussed in the following text.

Architect—The architect normally has responsibility for the following:

- The selection of the precast concrete panel finish, shape, and size.

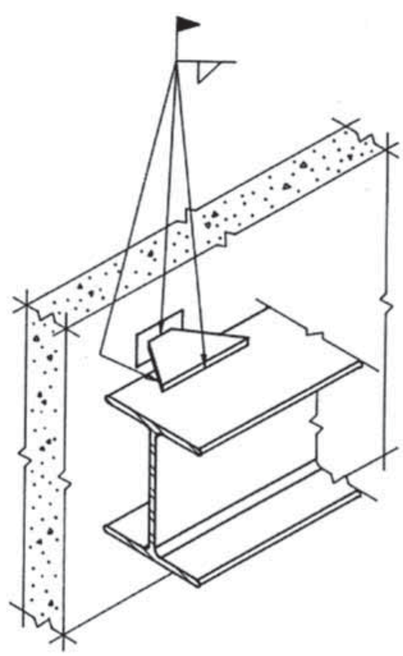


Fig. 8-4. Seismic shear plate connection to spandrel. (Taken from reference Architectural Precast Concrete, *Second Edition, PCI. Used with permission Precast/Prestressed Concrete Institute.)*

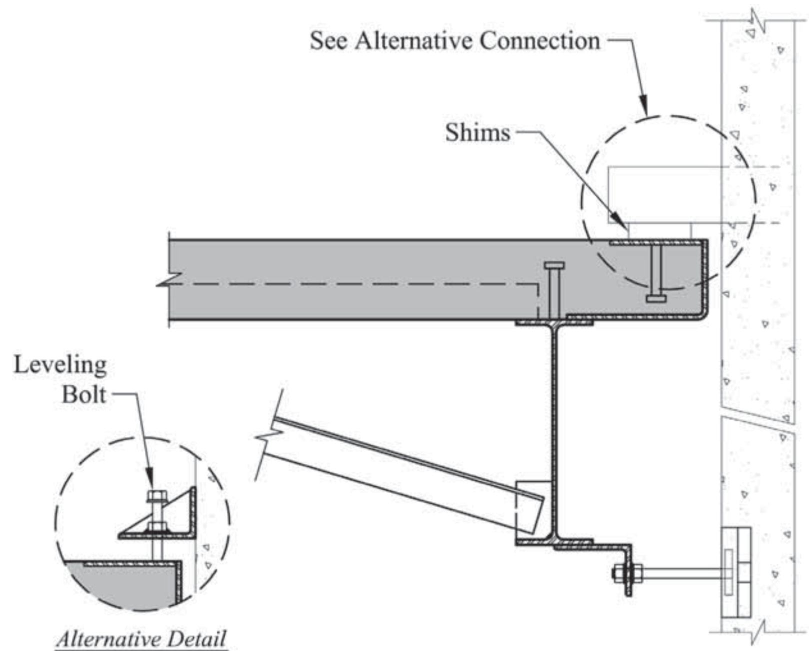


Fig. 8-5. Vertical alignment methods. (Adapted from reference Architectural Precast Concrete, *Second Edition, PCI. Used with permission Precast/Prestressed Concrete Institute.)*

- The performance of the precast concrete wall system with respect to moisture and thermal protection of the building.
- The selection of the wall system type (for example, conventional panel with insulation affixed to the interior surface, an independent insulated partition wall immediately interior to the concrete panel, or a sandwich wall with insulation between an exterior concrete wythe and an interior structural concrete wythe).
- The location of horizontal soft joints (and thus the location of the system of support framing).
- The dimension of the horizontal soft joint and performance of sealant joint (usually in consultation with the SER to understand practical limits for controlling movement due to the steel frame and support assembly).
- The selection of the precast panel wall thickness and structural performance requirements (usually in consultation with the SER).
- The precast panel plan location with respect to column and spandrel centerlines.
- The selection of the strategy for supporting the precast panels (usually in consultation with the SER).
- The setting of acceptable tolerances for erected and adjusted location of the precast wall plane.

Structural Engineer of Record (SER)—The SER normally has responsibility for the following:

- The design of the primary building structure, including the slab, slab edge detail, column, spandrel beam, roll beams, kickers, embedded bearing plates, etc., to support the forces imposed by the precast concrete panel system with due consideration to stiffness requirements.
- Providing the architect with structural design deformations for use in designing the soft joints.
- Providing the architect with the vertical and horizontal in-plane movements that the precast panel connections must accommodate for structural deformations for inclusion in the precast concrete specifications and/or design.
- Providing for field adjustable items at the slab edge accommodating some, if not all, of the frame erection tolerance, with the amount of adjustability coordinated with the requirements in the precast concrete specification.
- Indicating on the structural drawings pertinent assumptions and limitations about the loads from the precast concrete panels.
- Indicating load support points on the contract drawings.
- The review and approval of shop drawings and field erection drawings for the effect of precast panels and attachments on the primary building structure.

Precast Manufacturer and SSE—The precast manufacturer and SSE normally have responsibility for the following:

- The precast concrete panel design, including all reinforcement.
- The precast panel bearing and lateral connection design, including all supplemental hangers, kickers, and other structural steel elements required to support the panels.
- Providing for field adjustment as necessary to address fabrication and erection tolerances.
- The preparation of submittals, including shop drawings and field erection drawings.

8.5 CONNECTION TYPES

An efficient precast concrete panel system requires insight and early planning with regard to erection procedures in addition to the manufacturing process. Connections designed considering erection and fabrication processes tend to be the most economical. It is important that the precast manufacturer be involved early in the design process, providing insight regarding strategies toward connection types and layout.

Connections to the supporting structure can be made in a variety of manners. Common hardware includes inserts, bolts, structural steel, embedded plates with welded headed studs, threaded rods, and reinforcing bars—with inserts being the most common choice. The strength of a threaded insert depends greatly on its levelness, angle of orientation, and embedment. Wedge inserts are sometimes used in conditions where the design loads are small, but are not typically used in areas of high seismicity.

As seen in Figure 8-6, panel connection schemes typically consist of two bearing connections and a minimum of two tie-back connections. Bearing connections transfer vertical and out-of-plane lateral loads to the primary structure, and should be level with one another. Typically, bearing connections for spandrel-type panels are located near the floor level and are, therefore, near the middle to top of the panel. Solid wall-type panels more frequently bear near the bottom of the panel. Tie-back connections, which provide only out-of-plane lateral restraint, are usually located on the opposite edge of the panel.

Bearing Connections—These can be either direct or eccentric. Direct bearing connections are located nearly in line with the center of mass of the precast panel, whereas the

precast panel is offset from the bearing support in eccentric connections.

Direct bearing connections with threaded inserts or shims are common where precast panels are supported by the foundation wall. Alternatively for façade-supported conditions, Figure 8-7 demonstrates a direct bearing strategy for a hung façade support in which the precast panel is in direct bearing on a column bracket. In a case like this, although the panel mass is centered directly on the bearing surface, the eccentricity of the load with respect to the column must still be accounted for. Most hung façade supports, whether in direct or eccentric bearing, will contain loading eccentricities that must be resolved with respect to the primary structure.

Eccentric bearing connections are common at spandrel beam and column supports. Typical eccentric bearing schemes are detailed in Figures 8-8 and 8-9. Multiple variations on this scheme are possible, using angles, wide flange sections, channels, hollow structural sections, and flat bars.

Tie-Back Connections—These are not intended to provide any vertical restraint, nor in-plane lateral restraint. Tie-back connections resist wind and seismic loads only in the direction perpendicular to the plane of the panel. Common tie-back connection details are shown in Figure 8-10. Architectural panel layouts sometimes produce unique tie-back conditions where typical access to the connection is not available. Figure 8-11 offers strategies to provide a tie-back connection in the condition where the tie-back must be located above the bottom of the spandrel beam flange or within the flanges of a column section.

The SER must detail the supporting primary structure for the localized bending effects that are imposed from tie-back connections. The application of additional stiffener plates, heavier sections, and kickers are common methods of accounting for these stresses.

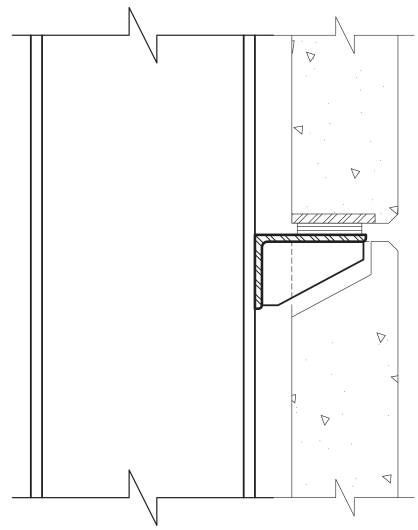


Fig. 8-7. Direct bearing connection to building columns. (Adapted from reference Architectural Precast Concrete, *Second Edition, PCI. Used with permission Precast/Prestressed Concrete Institute.)*

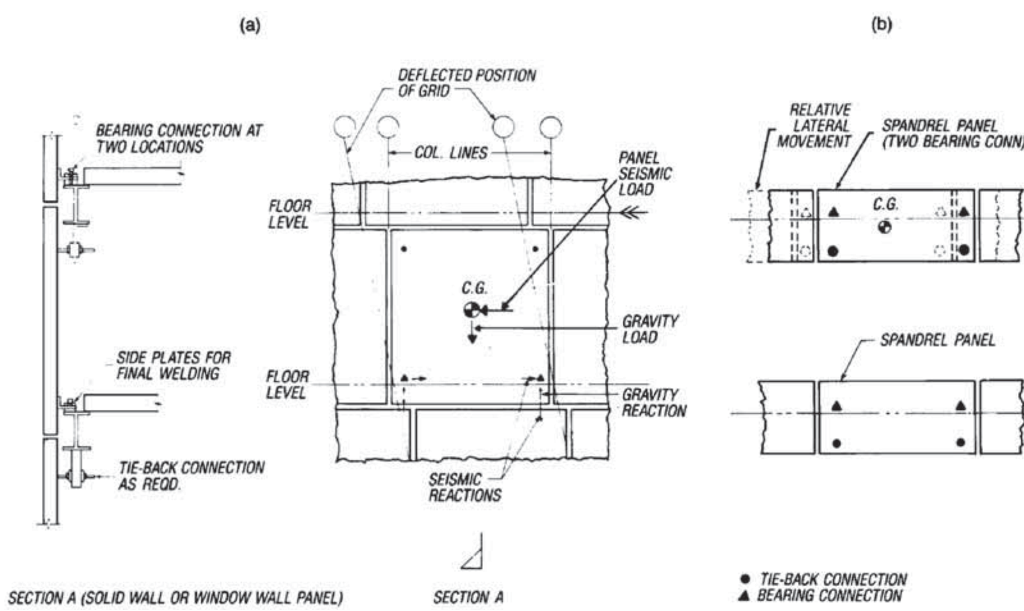


Fig. 8-6. Panel connection concepts. (Taken from reference Architectural Precast Concrete, *Second Edition, PCI. Used with permission Precast/Prestressed Concrete Institute.)*

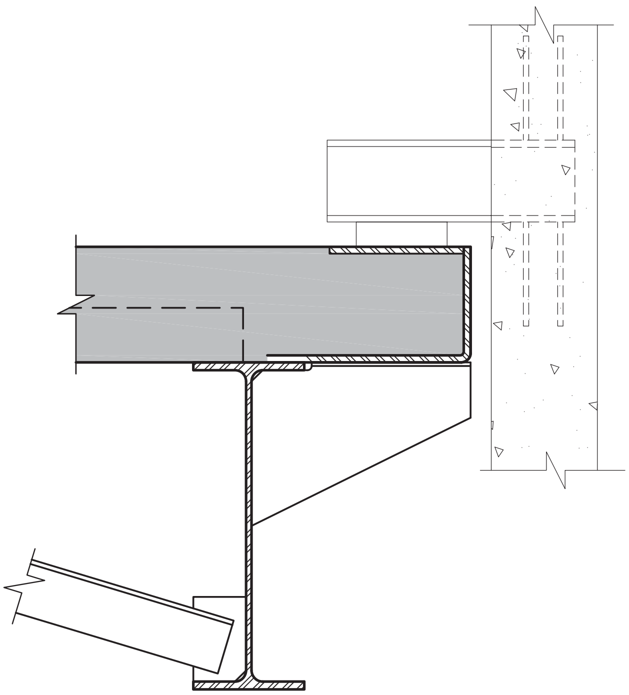


Fig. 8-8. Eccentric bearing connection to structural slab. (Adapted from reference Architectural Precast Concrete, *Second Edition, PCI. Used with permission Precast/Prestressed Concrete Institute.)*

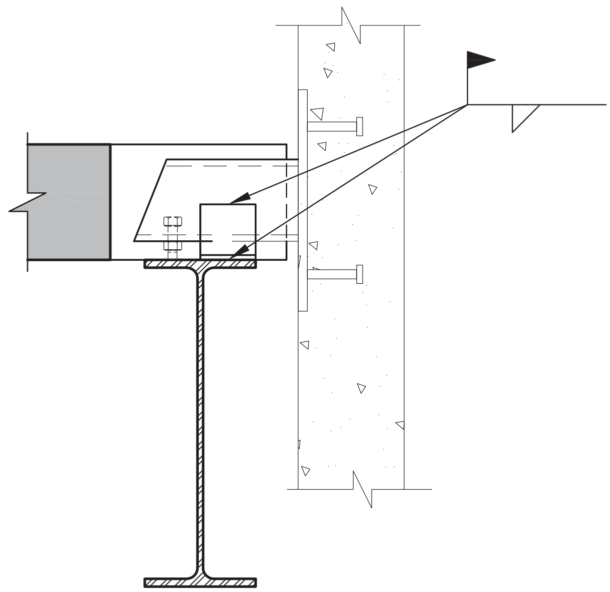


Fig. 8-9. Eccentric bearing connection to spandrel beam. (Adapted from reference Architectural Precast Concrete, *Second Edition, PCI. Used with permission Precast/Prestressed Concrete Institute.)*

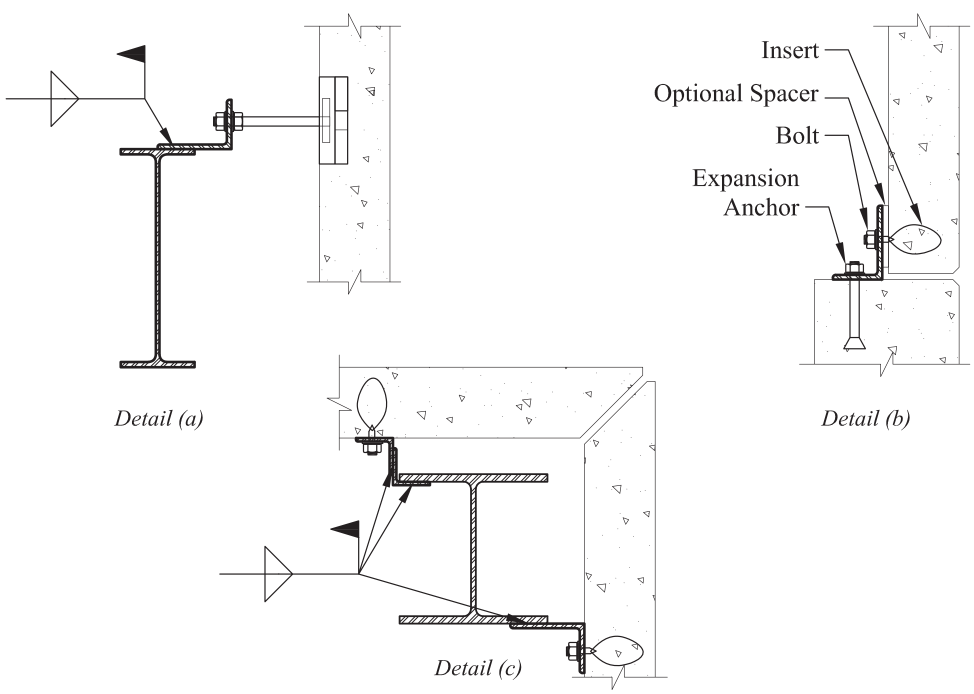


Fig. 8-10. Bolted tie-backs. (Adapted from reference Architectural Precast Concrete, *Second Edition, PCI. Used with permission Precast/Prestressed Concrete Institute.)*

Alignment Connections—These are typically used to adjust the position of individual precast panels relative to each other. Common bolted and welded alignment details are shown in Figures 8-12 and 8-13, respectively.

The SER should clearly specify the location and type of each connection on the contract drawings. The direction and approximate magnitude of applied loads on the connections to be designed by the precast engineer also should be identified.

8.6 COLUMN-SUPPORTED STORY-TALL PANEL

Column-supported story-tall precast panels are perhaps the most efficient façade-supported precast system. These panels typically bear on steel brackets and have one bearing connection and one tie-back connection at each column. See Figure 8-14.

The efficiency in column-supported story-tall panels is in the directness of the load paths. Panel connections are located near the floor levels, which minimizes flexural effects on the columns. Eccentricities typically have little effect on the column size.

8.7 COLUMN-SUPPORTED SPANDREL PANEL

The most efficient location for the vertical support of precast concrete spandrel panels is at the columns. Columns can easily be designed to resist the eccentricity of the panel weight. The precast spandrel panel is designed to span between columns for gravity forces, but may or may not be designed to span between columns for the out-of-plane wind and seismic forces. Supplemental out-of-plane supports can be provided to the slab if required for out-of-plane forces.

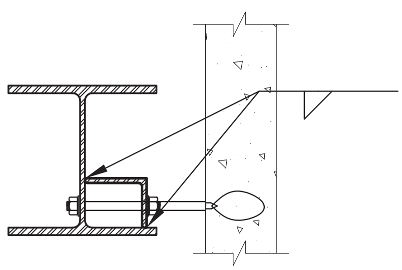
Connections should be located to minimize torsional stresses due to the "twist" that occurs if a tie-back force is offset from the center of rigidity of the supporting element. Figure 8-10a shows an example of an offset connection that should be avoided with heavy panels or high lateral loads, as this connection will tend to twist the column. Figure 8-11 includes alternative tie-back connections that may reduce this effect on the column.

Column-supported spandrel panels typically bear on steel brackets near the top of the panel, and have tie-back connections near the bottom of the panel. Thus, one bearing connection and one tie-back connection is made at each column.

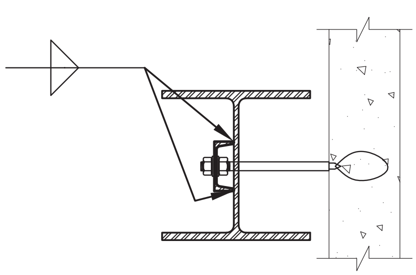
Example 8.1 outlines the concepts the SER may use to develop the forces to the primary structure from the precast wall panels. Although the SSE will design the panels and their attachments, Example 8.1 demonstrates the work that the SER must perform to design the primary structure and determine what the drawings will indicate.

8.8 SPANDREL-SUPPORTED STORY-TALL PANEL

Spandrel-supported story-tall precast panels typically bear on the slab edge at the floor level near the bottom of the panel and have tie-back connections located near the top of the panel at the upper floor level. A typical method of attachment is illustrated in Figure 8-6a. The eccentricity of the façade-supported system is not nearly as significant as with spandrel panels described in the next section. A story-tall panel spreads the overturning eccentricity over a full story, resulting in more manageable stabilizing reactions. Figure 8-15 shows typical connections.

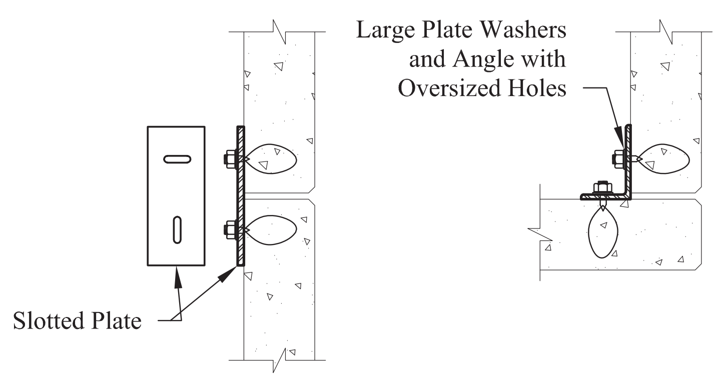


(a) Limited Access Around Beam/Column



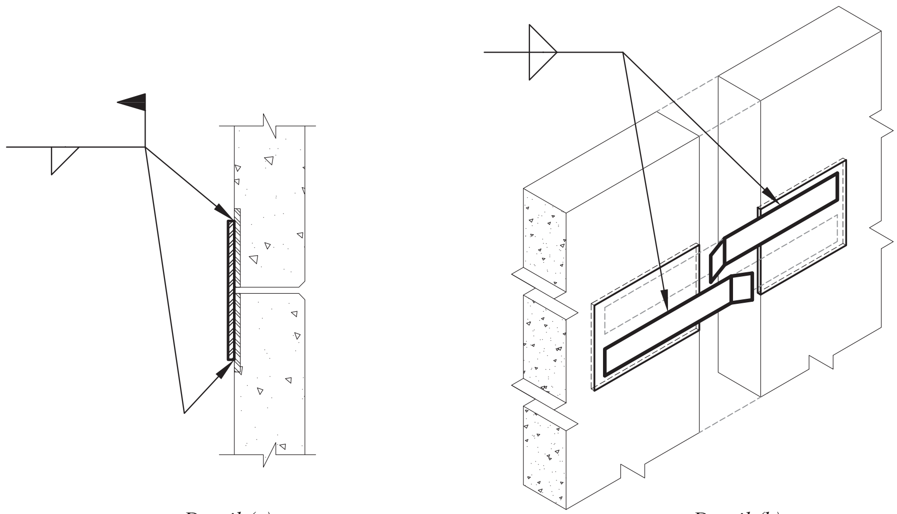
(b) No Access Between Panel and Beam/Column

Fig. 8-11. Tie-backs with little or no access around beam/column. (Adapted from reference Architectural Precast Concrete, *Second Edition, PCI. Used with permission Precast/Prestressed Concrete Institute.)*



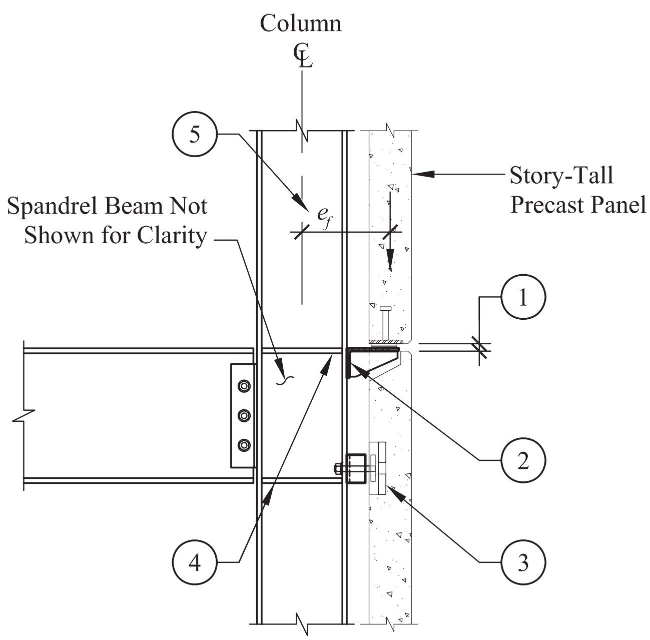
Detail (a) *Detail (b)*

Fig. 8-12. Bolted alignment.



Detail (a) *Detail (b)*

Fig. 8-13. Welded alignment.
(Adapted from reference Architectural Precast Concrete, *Second Edition, PCI.*
Used with permission Precast Prestressed Concrete Institute.)



NOTES:

1. Joint to allow differential vertical movement. Shim stack (or leveling bolt) bearing support at panel joint.

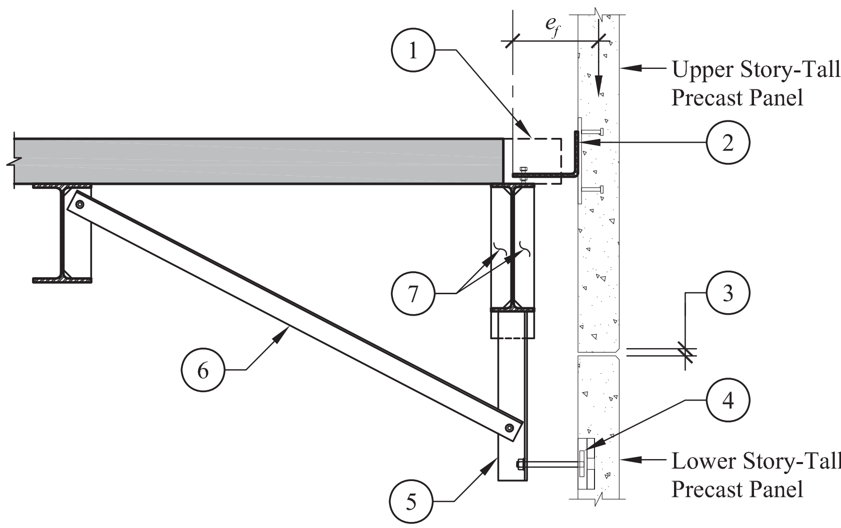
2. Steel bracket bearing connection. Typically designed by the SSE.

3. Tie-back connection at top of lower panel to allow vertical and horizontal relative movement.

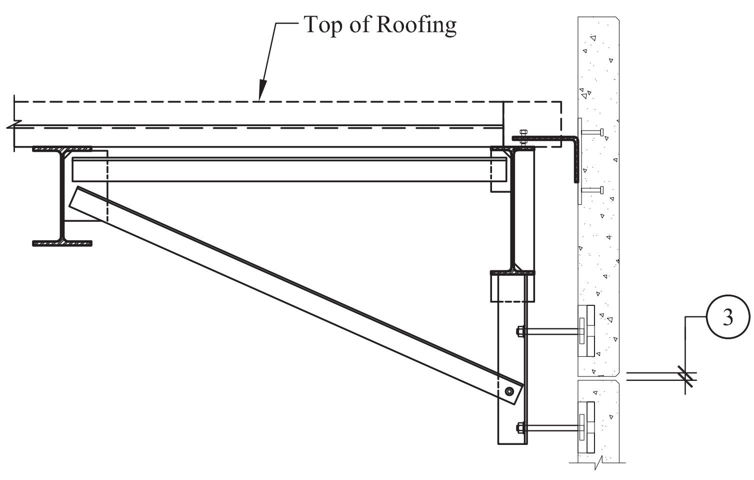
4. Stiffeners if required. Consider impact of stiffeners on the out-of-plane spandrel beam connection.

5. Maximum allowable eccentricity, e_f, specified by the Structural Engineer of Record.

Fig. 8-14. Typical column-supported story-tall precast panel.



(a) Detail at Composite Deck Floor Slab



(b) Detail at Metal Deck Supported Roof

NOTES:

1. Blockout in slab for bearing connection.
2. Bearing clip and connection to precast is typically specified by the SSE. Connection to panel is designed for moment to resolve eccentricity e_f.
3. Panel joint allows relative movement between panels.
4. Slotted (or threaded) insert tie-back connection allows vertical and in-plane relative movement.
5. Steel shape hanger from spandrel to receive tie-back connection.
6. Steel shape kicker to resist tie-back alone and avoid torsion on spandrel.
7. Stiffener plates, if required, for local flange bending for bearing clip on hanger.

Fig. 8-15. Typical spandrel-supported story-tall precast panel.

Examples 6.5 and 6.6 illustrate the procedures required to design spandrel beams for the effects of eccentric loading due to typical story-tall precast panels.

8.9 SPANDREL-SUPPORTED SPANDREL PANEL

Spandrel precast panels are often utilized in architectural layouts with continuous horizontal bands of windows. While it tends to be more efficient to support the panels at the columns, spandrel beam support is also common. Spandrel-supported spandrel panels are typically subject to significant eccentricities resulting in the added costs of reinforcement or heavier steel sections. For this reason, the designer should minimize the distance between the panel and the support member.

Providing bearing connections directly over the centerline of the spandrel beam minimizes the effect of eccentricity on the primary structure, but is often impractical because of the length of the connection required and the sizable eccentricities imparted on the connection to the panel (see Figure 8-16a). It is more common to bear on the slab edge or steel bracket supports such that the panel connection is located as close as possible to the inside face of the precast panel. This is demonstrated in Figure 8-16b. When the bearing connection occurs on a cantilevered slab edge, additional support, such as stiffener plates or angles, is typically provided under the bearing points to limit rotation and deflection, as shown in Figure 8-16. The eccentricity to the spandrel beam must be considered in the design of the spandrel beam.

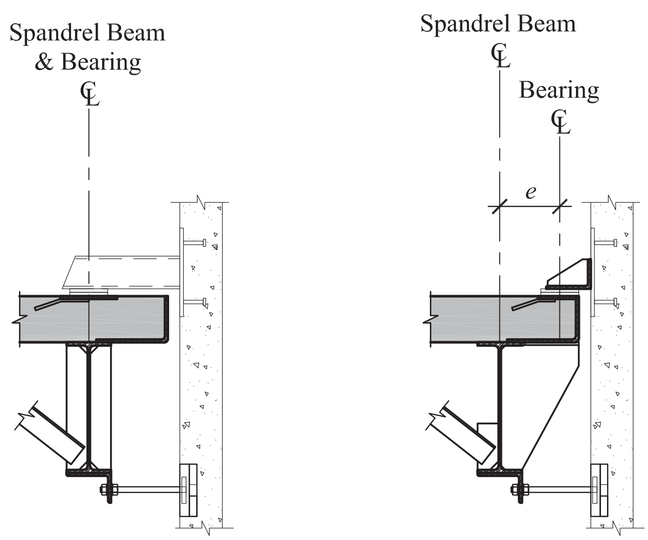
Spandrel-supported spandrel panels typically have two connections located near the top of the panel that bear on the structure at the floor level. Tie-back connections are often located near the bottom of the panel height and attach either to the bottom flange of the spandrel beam or to a hanger beneath the spandrel beam. Where required, additional tie-back connections are provided for out-of-plane lateral load resistance, and shear plates can be added for in-plane lateral load resistance. This is more common in regions of high seismicity.

8.10 POTENTIAL PROBLEMS WITH SUPPORT AND ANCHORAGE OF PRECAST CONCRETE WALL PANELS

The following is a list of potential problems that designers should be aware of and avoid when designing the support and anchorage systems for precast concrete wall panels. There is no particular meaning implied by the order of the list.

- Lack of coordination in the erection sequence for brackets, blockouts, and embedment plates. For example, if attachments to the primary structure include connections to steel elements that are embedded in concrete slabs, the construction sequence must be coordinated such that the slabs have been poured and cured to adequate design strength prior to precast erection.

- Cantilevers with insufficient stiffness may deflect or rotate significantly during erection. Stiff cantilevers are beneficial in reducing the amount of final adjustment required after erection.



(a) Direct Bearing Connection Over Spandrel (Impractical Length and Cost)

(b) Eccentric Bearing Connection (More Practical and Common)

Fig. 8-16. Direct vs. eccentric bearing connections at spandrels.

- Lack of clarity in the division of responsibilities for designing and providing attachment and support components. Responsibility for the design of miscellaneous angles, embedment plates, and similar items must be clearly indicated in the contract documents.

- Inadequate coordination of joints in architectural elevations with the points of load application to the primary structure as anticipated by the SER.

- Unanticipated loads or moments on the primary structure from attachment details designed by the SSE.

- Unanticipated rotations and flexibility of the precast support from kickers that resolve eccentricity into lightweight roof elements, such as steel joists, not designed for the resulting loads.

- Unanticipated spandrel loading from tie-back connections attached to the bottom flange of spandrel beams without consideration of the effects on the spandrel beams.

- Inadequate coordination and accommodation for adjustability, possibly also resulting in greater eccentricities than anticipated in the attachment designs provided by the SSE.

Example 8.1—Precast Concrete Panel Supported on a Steel-Framed Building

For a steel-framed building located in a seismic zone that is clad with concrete panels as shown in Figure 8-17, determine the dead loads that the precast panel imposes on the attachment points to the building structure and the seismic forces that the precast panel imposes on the building at each of its attachment points using the 2006 *International Building Code* (ICC, 2006).

Given:
The building is located in a Seismic Design Category C such that the seismic coefficient, S_{DS} = 0.28. The gravity load of the panel is resisted e_p = 10 in. from the exterior face of the panel. The average roof height of the building, h = 50 ft above grade and the panels are attached to the structure at z = 30 ft above grade.

The panels are constructed with normal weight concrete, w_c = 150 pcf. The height of one panel, h_p = 6½ ft, the width, b_p = 1 ft, the length, L_p = 30 ft, and the thickness, t_p = 6 in. = ½ ft. There are strip windows above and below the panels at a height, h_w = 6 ft, that weigh, w_g = 10 psf. The panels are attached directly to the structural steel frame. There are no shallow embedments into the concrete.

The horizontal distance between the connections at the ends of the panel, L_c = 29 ft. The vertical distance between the panel attachment points, y_p = 30 in. and the vertical distance between the bottom of the panel and tieback points, y_{tb} = 24 in. The distance between the exterior face of the panel and the glazing, e_w = 10 in. The connections to the building and their associated restraints are shown in Figure 8-18.

From ASCE 7-05, the component importance factor from Section 13.1.3, I_p = 1.0, the component amplification factor from Table 13.5-1, a_p = 1.0, and the component response modification factor from Table 13.5-1, R_p = 2.5.

Solution:
The dead loads that the precast panel imposes on the attachment points to the building structure are shown in Figure 8-19.

The cross-sectional area of the panel is,

$$\begin{aligned} A_p &= \left[2b_p + \left(h_p - 2t_p\right)\right]t_p \\ &= \left[2\left(1\text{ ft}\right) + \left(6\tfrac{1}{2}\text{ ft} - 2\left(\tfrac{1}{2}\text{ ft}\right)\right)\right]\left(\tfrac{1}{2}\text{ ft}\right) \\ &= 3.75\text{ ft}^2 \end{aligned}$$

The centroid of the panel with respect to the exterior face is,

$$\begin{aligned} x_p &= \frac{\dfrac{2t_p b_p^2}{2} + \dfrac{\left(h_p - 2t_p\right)t_p^2}{2}}{3.75\text{ ft}^2} \\ &= \left[\frac{\dfrac{2\left(\tfrac{1}{2}\text{ ft}\right)\left(1\text{ ft}\right)^2}{2} + \dfrac{\left(6\tfrac{1}{2}\text{ ft} - 2\left(\tfrac{1}{2}\text{ ft}\right)\right)\left(\tfrac{1}{2}\text{ ft}\right)^2}{2}}{3.75\text{ ft}^2}\right] \\ &\quad \times 12\text{ in./ft} \\ &= 3.80\text{ in.} \end{aligned}$$

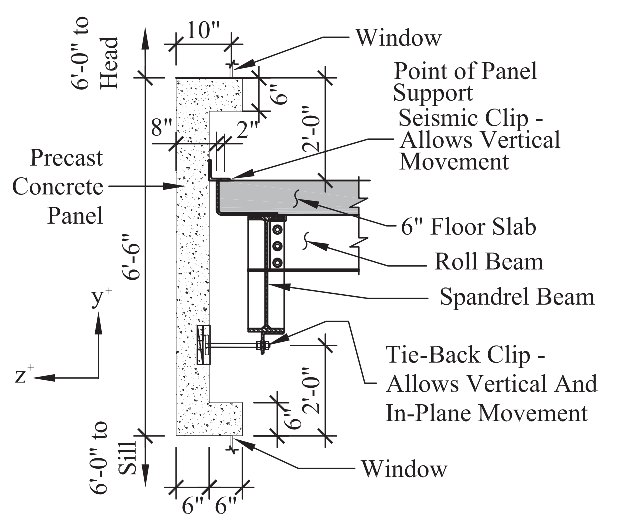


Fig. 8-17. Section of spandrel beam supporting precast concrete panel.

The weight of the panel is,

$$\begin{aligned} W_{pnl} &= w_c A_p L_p \\ &= 0.150 \text{ kip/ft}^3 \left(3.75 \text{ ft}^2\right)\left(30 \text{ ft}\right) \\ &= 16.9 \text{ kips} \end{aligned}$$

The weight of the windows above bearing on the panel is,

$$\begin{aligned} W_{wa} &= w_g h_w L_p \\ &= 0.010 \text{ kip/ft}^2 \left(6 \text{ ft}\right)\left(30 \text{ ft}\right) \\ &= 1.80 \text{ kips} \end{aligned}$$

The weight of the windows below the panel is,

$$\begin{aligned} W_{wb} &= w_g h_w L_p \\ &= 0.010 \text{ kip/ft}^2 \left(6 \text{ ft}\right)\left(30 \text{ ft}\right) \\ &= 1.80 \text{ kips} \end{aligned}$$

The total dead load transmitted to the structure is,

$$\begin{aligned} W_D &= W_{pnl} + W_{wa} \\ &= 16.9 \text{ kips} + 1.80 \text{ kips} \\ &= 18.7 \text{ kips} \end{aligned}$$

The centroid of the dead load with respect to the point of the gravity support is,

$$\begin{aligned} z_D &= e_p - \frac{W_{pnl} x_p + W_{wa} e_w}{W_D} \\ &= 10 \text{ in.} - \frac{16.9 \text{ kips}\left(3.80 \text{ in.}\right) + 1.80 \text{ kips}\left(10 \text{ in.}\right)}{18.7 \text{ kips}} \\ &= 5.60 \text{ in.} \end{aligned}$$

The vertical dead load to be resisted at points A and C is,

$$\begin{aligned} P_{DY} &= \frac{W_D}{2} \\ &= \frac{18.7 \text{ kips}}{2} \\ &= 9.35 \text{ kips} \end{aligned}$$

The horizontal (out-of-plane) dead load to be resisted at points A, C, D, and F is,

$$\begin{aligned} P_{DZ} &= \frac{W_D}{2}\left(\frac{z_D}{y_p}\right) \\ &= \frac{18.7 \text{ kips}}{2}\left(\frac{5.60 \text{ in.}}{30 \text{ in.}}\right) \\ &= 1.75 \text{ kips} \end{aligned}$$

To determine the seismic forces that the precast panel imposes on the building at each of its attachment points, the 2006 IBC, Section 1613 refers the reader to ASCE 7-05. See Figure 8-20.

The total effective seismic weight of the panel and windows includes the dead load of the panel and the weight of the portions of the windows above and below that are tributary to the panel,

$$\begin{aligned} W_p &= W_{pnl} + \frac{W_{wa}}{2} + \frac{W_{wb}}{2} \\ &= 16.9 \text{ kips} + \frac{1.80 \text{ kips}}{2} + \frac{1.80 \text{ kips}}{2} \\ &= 18.7 \text{ kips} \end{aligned}$$

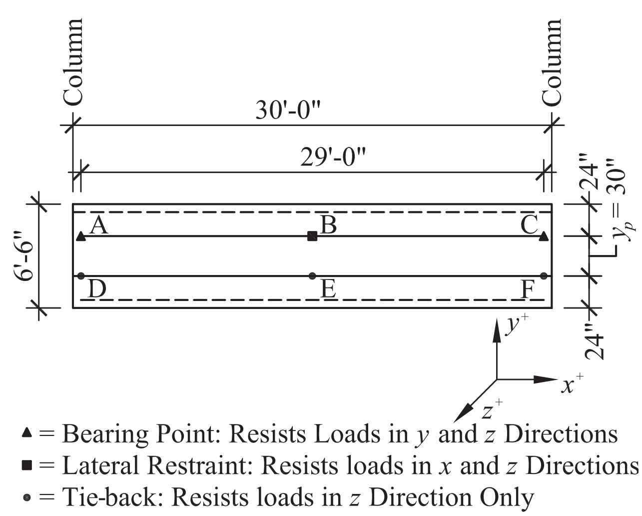


Fig. 8-18. Elevation of precast panel.

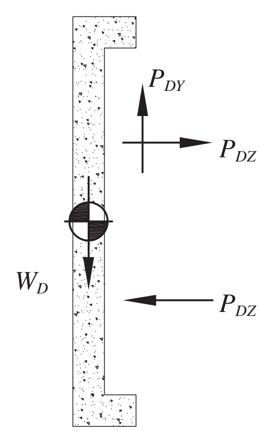


Fig. 8-19. Free body diagram of dead load forces on precast panel.

The centroid of the seismic load with respect to the point of the gravity support is,

$$z_E = e_p - \frac{W_{pnl}\,x_p + \left(\dfrac{W_{wa}}{2} + \dfrac{W_{wb}}{2}\right)e_w}{W_p}$$

$$= 10 \text{ in.} - \frac{16.9 \text{ kips}\left(3.80 \text{ in.}\right)}{18.7 \text{ kips}}$$

$$+ \frac{\left(\dfrac{1.80 \text{ kips}}{2} + \dfrac{1.80 \text{ kips}}{2}\right)10 \text{ in.}}{18.7 \text{ kips}}$$

$$= 5.60 \text{ in.}$$

The centroid of the seismic load with respect to the edge corner of the panel is,

$$y_E = \frac{W_{pnl}\left(\dfrac{h_p}{2}\right) + W_{wa}\left(\dfrac{h_p}{2}\right)}{W_p}$$

$$= \frac{16.9 \text{ kips}\left(\dfrac{6\tfrac{1}{2} \text{ ft}}{2}\right) + 1.80 \text{ kips}\left(\dfrac{6\tfrac{1}{2} \text{ ft}}{2}\right)}{18.7 \text{ kips}}$$

$$\times 12 \text{ in./ft}$$

$$= 39.0 \text{ in.}$$

From ASCE 7-05 Equation 13.3-1:

$$F_p = \frac{0.4 a_p S_{DS} W_p I_p}{R_p}\left(1 + \frac{2z}{h}\right)$$

$$= \frac{0.4\left(1.0\right)\left(0.28\right)\left(18.7 \text{ kips}\right)\left(1.0\right)}{2.5}\left[1 + \frac{2\left(30 \text{ ft}\right)}{50 \text{ ft}}\right]$$

$$= 1.84 \text{ kips}$$

From ASCE 7-05 Equation 13.3-2:

$$\begin{aligned} F_{p\,max} &= 1.6 S_{DS} I_p W_p \\ &= 1.6\left(0.28\right)\left(1.0\right)\left(18.7 \text{ kips}\right) \\ &= 8.38 \text{ kips} \end{aligned}$$

From ASCE 7-05 Equation 13.3-3:

$$\begin{aligned} F_{p\,min} &= 0.3 S_{DS} I_p W_p \\ &= 0.3\left(0.28\right)\left(1.0\right)\left(18.7 \text{ kips}\right) \\ &= 1.57 \text{ kips} \end{aligned}$$

Equation 13.3-1 controls and this value is used to proceed with the calculations. The seismic force, F_p, must be considered acting both parallel to the plane of the wall and perpendicular to the plane of the wall.

Note that these values are for the panel itself, the body of the wall panel connections, and the forces on the base building structure. Based on ASCE 7-05 Equation 13.3-1 the forces on the fasteners of the connecting system may be as much as 313% of the force on the panel, F_p.

The seismic loads parallel to the plane of the panel are illustrated in Figure 8-21.

Note that connection B, shown in Figure 8-20, must take all of the in-plane shear from the panel, and connections A and C resist the moment associated with the vertical eccentricity between the seismic force and the supports.

The in-plane shear at point B is,

$$P_{xB} = F_p = 1.84 \text{ kips}$$

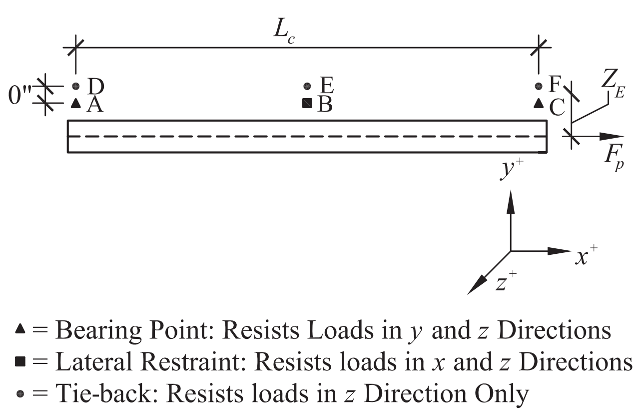


Fig. 8-20. Plan view of precast panel.

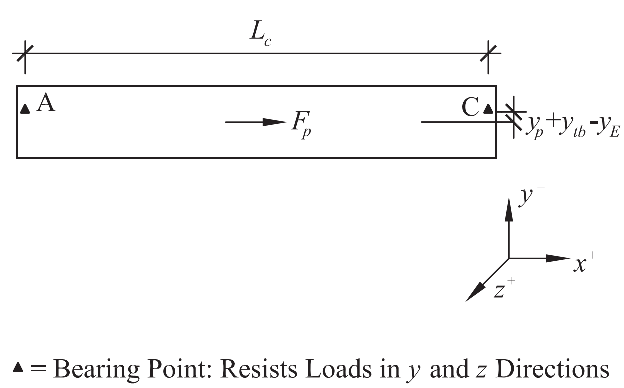


Fig. 8-21. Elevation view of panel.

The vertical load at points A and C due to the in-plane seismic load is,

$$\begin{aligned}P_{yAC} &= \frac{F_p\left(y_p + y_{tb} - y_E\right)}{L_c}\\ &= \frac{1.84\text{ kips}\left(30\text{ in.} + 24\text{ in.} - 39.0\text{ in.}\right)}{29\text{ ft}\left(12\text{ in./ft}\right)}\\ &= 0.0793\text{ kip}\end{aligned}$$

The out-of-plane load at points A, C, D, and F due to the in-plane seismic load acting at eccentricity z_E is,

$$\begin{aligned}P_{zACDF} &= \frac{F_p z_E}{2L_c}\\ &= \frac{1.84\text{ kips}\left(5.60\text{ in.}\right)}{2\left(29\text{ ft}\right)\left(12\text{ in./ft}\right)}\\ &= 0.0148\text{ kip}\end{aligned}$$

The out-of-plane load on the connections for seismic load perpendicular to the plane of the panel is illustrated in Figure 8-22.

The force, F_p, is assumed to be uniformly distributed along the length of the panel, L_p, acting at height y_E above the bottom of the panel. Using AISC Manual Table 3-22c, assuming that the panel behaves like a continuous beam with two equal spans, the center connections (B and E) must cumulatively resist approximately $\tfrac{5}{8}F_p$, and the end connections (A, C, D, and F) must cumulatively resist $\tfrac{3}{8}F_p$.

Assuming that the panel is simply supported between the top and bottom rows of the connections, the rows of connections must resist the following proportions of the load F_p:

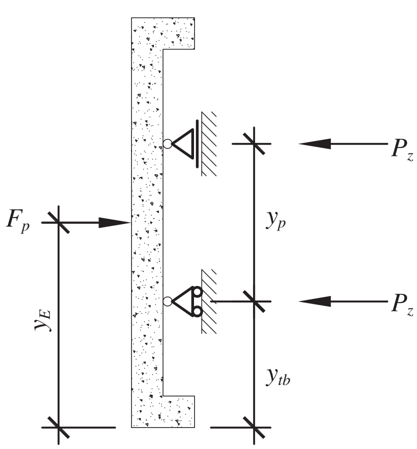


Fig. 8-22. Section at center of panel.

The proportion of the load resisted by the top row of the connections is,

$$\begin{aligned}C_{top} &= \frac{y_E - y_{tb}}{y_p}\\ &= \frac{39\text{ in.} - 24\text{ in.}}{30\text{ in.}}\\ &= 0.500\end{aligned}$$

The proportion of the load resisted by the bottom row of the connections is,

$$\begin{aligned}C_{top} &= \frac{y_p + y_{tb} - y_E}{y_p}\\ &= \frac{30\text{ in.} + 24\text{ in.} - 39\text{ in.}}{30\text{ in.}}\\ &= 0.500\end{aligned}$$

The out-of-plane load resisted by connection B is,

$$\begin{aligned}P_{zB} &= C_{top}\left(\tfrac{5}{8}F_p\right)\\ &= 0.500\left(\tfrac{5}{8}\right)\left(1.84\text{ kips}\right)\\ &= 0.575\text{ kip}\end{aligned}$$

The out-of-plane load resisted by connection E is,

$$\begin{aligned}P_{zE} &= C_{bot}\left(\tfrac{5}{8}F_p\right)\\ &= 0.500\left(\tfrac{5}{8}\right)\left(1.84\text{ kips}\right)\\ &= 0.575\text{ kip}\end{aligned}$$

The out-of-plane load resisted by connections A and C is,

$$\begin{aligned}P_{zAC} &= C_{top}\left(\tfrac{3}{8}F_p\right)\\ &= 0.500\left(\tfrac{3}{8}\right)\left(1.84\text{ kips}\\ &= 0.345\text{ kip}\end{aligned}$$

The out-of-plane load resisted by connections D and F is,

$$\begin{aligned}P_{zDF} &= C_{bot}\left(\tfrac{3}{8}F_p\right)\\ &= 0.500\left(\tfrac{3}{8}\right)\left(1.84\text{ kips}\right)\\ &= 0.345\text{ kip}\end{aligned}$$

The following tables summarize the dead and seismic forces at each connection point:

Service Level Forces due to Dead Loads on Building Structure, kips			
Point	Direction of Force		
	x	*y*	*z*
A	n/a	9.35	+1.75
B	0	n/a	0
C	n/a	9.35	+1.75
D	n/a	n/a	–1.75
E	n/a	n/a	0
F	n/a	n/a	–1.75

In-Plane Seismic Forces On Building Structure, kips			
Point	Direction of Force		
	x	*y*	*z*
A	n/a	± 0.0793	±0.0148
B	±1.84	n/a	0
C	n/a	±0.0793	±0.0148
D	n/a	n/a	±0.0148
E	n/a	n/a	0
F	n/a	n/a	±0.0148

Out-of-Plane Seismic Forces on Building Structure, kips			
Point	Direction of Force		
	x	*y*	*z*
A	n/a	0	±0.345
B	0	n/a	±0.575
C	n/a	0	±0.345
D	n/a	n/a	±0.345
E	n/a	n/a	±0.575
F	n/a	n/a	±0.345

Comments:

This example shows how to calculate the seismic loads that a precast concrete panel imposes on a building. Note that although the wind loads on the panel are not calculated in this example, they may control the design of the attachment of the facade to the building. The calculations for the wind loads are similar to those presented for seismic out-of-plane loads.

In this example the attachment force is calculated at $z = 30$ ft. However, the designer may choose to group the facade connection designs and design all connections as if the connection occurred at the roof. In this case, using $z_{max} = 50$ ft:

$$F_{proof} = \frac{0.4a_p S_{DS} W_p I_p}{R_p}\left(1+\frac{2z_{max}}{h}\right)$$

$$= \frac{0.4(1.0)(0.28)(18.7\text{ kips})(1.0)}{2.5}\left(1+\frac{2(50\text{ ft})}{50\text{ ft}}\right)$$

$$= 2.51\text{ kips}$$

This compares to $F_p = 1.84$ kips determined previously using $z = 30$ ft.

Chapter 9
Aluminum Curtain Walls

9.1 GENERAL DESCRIPTION OF ALUMINUM CURTAIN WALL SYSTEMS

Aluminum curtain wall systems can be either custom or standard type. Standard type curtain walls are usually more economical, but custom-type curtain walls are commonly used because they are tailored to the architect's design and can be applied to buildings of all shapes and sizes. Whether custom- or standard-type walls are selected, the walls can be assembled in a variety of different systems. Some of the more common systems are: stick-built system, pre-assembled unit system, and unit-and-mullion system. These systems are illustrated in Figures 9-1 through 9-3, respectively.

Each type of system has different strategies of installation and varying degrees of adjustment. Stick systems are installed one piece at a time, with the mullions typically installed first. Pre-assembled unit systems are panelized and designed to interlock with adjacent vertical mullions and horizontal rails. Unit-and-mullion systems are a mix between stick systems and pre-assembled unit systems. Mullions are erected first, and pre-assembled units are installed between the mullions. Systems assembled in the plant require fewer connections in the field, which can be cost effective, albeit with reduced adjustability of the system in the field.

Common components of aluminum curtain wall systems are mullions, horizontal rails, spandrel panels, vision glass, interior mullion trim, and anchors. Walls can either be in-filled between floors or run continuously past the outside face of the floor slab edge. In-filled systems between floor slabs are common for horizontal strip windows and storefront, whereas continuous systems outside of the floor slab edge are common for multi-story curtain walls.

Aluminum curtain walls are preferred by designers not only because they are light in weight, but also for ease of installation. Less time is typically required to install aluminum curtain walls than other common façade systems.

The performance of an aluminum curtain wall system is related to the quality of design, fabrication, and installation. Effects from miscellaneous items, such as swing-stage loads and window washers, should be incorporated into the design. Often the most important part of the design is anchorage to the primary building structure. Movement tolerances and required clearances are the most important considerations for the attachment design.

9.2 STRATEGIES FOR SUPPORT OF ALUMINUM CURTAIN WALLS

A strategy for supporting the aluminum curtain wall system starts with the architect's decision of system type. In-filled curtain walls, single-span curtain walls outside of the floor slab edge, continuous multi-story curtain walls with lateral support at floor slabs, and multi-story curtain walls without floor slab support all have different methods of attachment to the primary building structure.

In-filled systems span between floor slabs and typically bear on the lower floor with lateral support at the upper floor. Single-span curtain walls have a fixed connection at one end of the frame and a movable connection at the other end. Continuous multi-story curtain walls with lateral support at floor slabs are very common, and are typically supported with a fixed connection from the spandrel beam at either the primary end or top of the panel, and with movable connections at intermediate floor slabs and at the opposite end of the panel. Figure 9-4 illustrates a multi-story support condition. Multi-story curtain walls without floor slab support are common at building atriums, and are typically supported with a supplemental vertical or horizontal structural steel girt system, which usually provides only lateral support.

The most appropriate and efficient means of anchoring the curtain wall to the primary building structure depends on the type of curtain wall. The following suggestions provide strategies toward successful design:

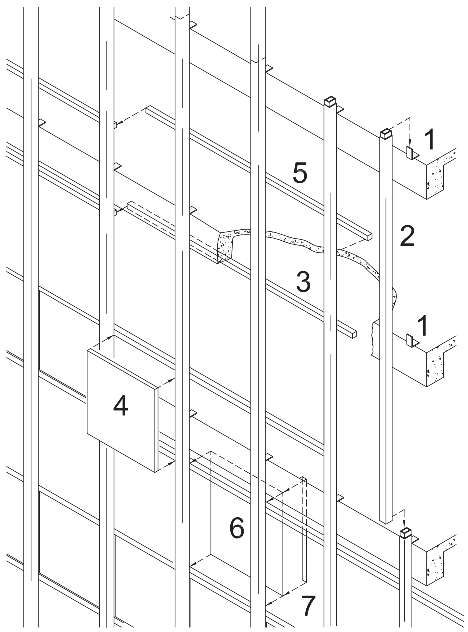
- Anchors should be located in an easily accessible location. Attachments to the top or side of the slab or spandrel beam are generally easier to install than underside connections.

- Adjustability in all connections should be provided in the vertical direction and both horizontal directions. Two-piece connections with both vertically and horizontally slotted holes are a common method of providing adjustability.

- It is preferable to limit the eccentricity on anchors.

- Since fireproofing is often applied prior to curtain wall installation, blockouts should be provided in the fireproofing, and patched after the anchors are attached.

- Bolt holes should be made in the shop; not field drilled or flame-cut. Erectors often prefer welded connections of steel pieces.

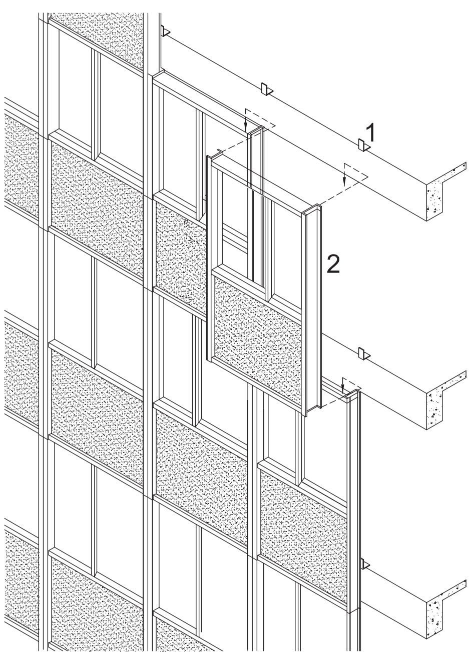


NOTES:

1. Anchors.
2. Mullion.
3. Horizontal rail (gutter section at window head).
4. Spandrel panel (may be installed from inside building).
5. Horizontal rail (window sill section).
6. Vision glass (installed from inside building).
7. Interior mullion trim.

Other Variations:
Mullion and rail sections may be longer or shorter than shown. Vision glass may be set directly in recesses in framing members, with applied stops, or in sub-frame, or may include operable sash.

Fig. 9-1. Stick system—schematic of typical version. (Taken from Curtain Wall Design Manual, *Editorial Revision: May 2005, AAMA. Used with permission, American Architectural Manufacturers Association.)*

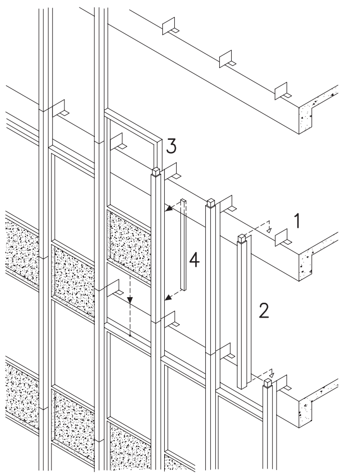


NOTES:

1. Anchors.
2. Pre-assembled framed unit.

Other variations:
Mullion sections may be interlocking "split" type or channel shapes with applied inside and outside joint covers. Units may be unglazed when installed or pre-glazed. Spandrel panel may be at either top or bottom of unit.

Fig. 9-2. Unit system—schematic of typical version. (Taken from Curtain Wall Design Manual*, Editorial Revision: May 2005, AAMA. Used with permission, American Architectural Manufacturers Association.)*

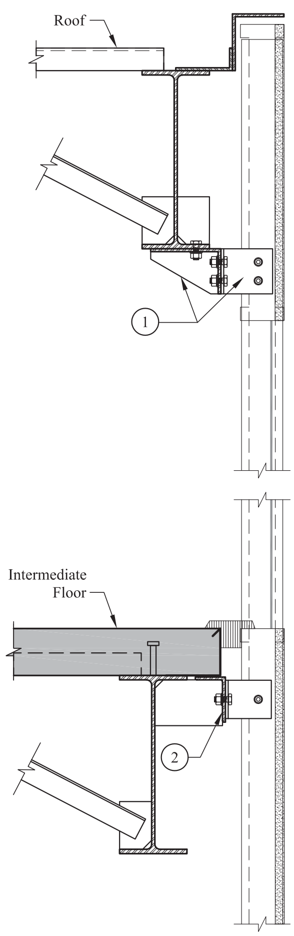


NOTES:

1. Anchors.
2. Mullion (either one- or two-story lengths).
3. Pre-assembled unit lowered into place behind mullion from above floor.
4. Interior mullion trim.

Other variations:
Framed units may be full-story height (as shown), either unglazed or pre-glazed, or separate spandrel cover units and vision glass units. Horizontal rail sections are sometimes used between units.

Fig. 9-3. Unit-and-mullion system–schematic of typical version. (Taken from Curtain Wall Design Manual*, Editorial Revision: May 2005, AAMA. Used with permission, American Architectural Manufacturers Association.)*



NOTES:

1. Dead-load attachment. Curtain wall hangs from roof structure in this example. Attachment detail must provide for both vertical and horizontal adjustments.
2. Wind-load attachment. Detail must provide out-of-plane support of mullion and allow vertical movement relative to structure.

Fig. 9-4. Multi-story curtain wall.

It is important that the installer understand the design intent of the various types of curtain wall attachments shown on the contract documents and shop drawings. Misunderstanding can lead to improper installation, and unintended restraint from improperly connected attachments can cause unanticipated forces on the system.

9.3 PARAMETERS AFFECTING THE DESIGN OF ALUMINUM CURTAIN WALL SUPPORTS

Parameters affecting the design of aluminum curtain wall systems include:

- Architectural decisions.
- Movement requirements.
- Field adjustability for tolerances and clearances.
- Durability.
- Fire-safing.

9.3.1 Architectural Decisions

Architectural decisions dictate the strategy for supporting the curtain wall system. Decisions that affect the support and attachment of the curtain wall system follow.

Location of the mullions and joints—The location of the vertical mullions and horizontal joints of the curtain wall system relative to the primary structural frame drives the designer's decision for how to support the wall. It is usually economical if the horizontal joints are located within the depth of the spandrel beam so the curtain wall can be attached to the supporting structure without the use of hangers. The closer the anchors are to the spandrel beams, the less distance the supporting attachment steel needs to extend, and this generally means less cost. If the mullion elevation is near the slab edge, it may be possible to fasten it directly to the slab.

Story height—The story height directly impacts the weight and thickness of the system, and also impacts the magnitude of the out-of-plane forces that must be transferred from the mullions to the primary structure.

Curtain wall/support cross section—Current curtain wall designs have numerous variations of cross sections with regard to the location of the glass relative to the mullions, rails, and structural support. Multi-story curtain walls in atriums often require horizontal support at intervals along the height of the curtain wall. Structural steel girts are often used to provide this support and can be integrated within, set in front of, or set behind the curtain wall. Another variation is curtain wall systems that span horizontally and require structural steel vertical support. These vertical girts can also be integrated within, set in front of, or set behind the glass.

9.3.2 Movement Requirements

Critical to the performance of an aluminum curtain wall system is the provision for allowance of movement in the joints and connections. Curtain walls are subject to a greater degree of thermal movement than most other types of façade systems, and inadequate detailing of connections to account for movement independent of the primary building structure can result in damage or potentially even failure of the wall system.

Thermal movement stems from both ambient air temperature and solar effects. The range in temperature of metal surfaces in curtain-wall frames has been known to be as much as 4 or 5 times greater than that of building frames, since temperatures of building frames do not usually vary significantly. It is therefore essential that thermal movement not be restrained at connections to the primary building structure.

The vertical and lateral stiffness of the supporting structure should satisfy the specifications of the curtain wall manufacturer. Common deflection limitations under lateral loads normal to the plane of the curtain wall are in the range of $L/175$, where L is the span of the framing member, or a maximum of ¾ in. The allowable deflection criterion is often $L/360$ for framing members that are connected to plaster or drywall surfaces. There is no industry rule of thumb for vertical stiffness of the supporting framing members, but limiting the support deflections to less than $L/360$ is usually sufficient for curtain wall systems.

9.3.3 Field Adjustability for Tolerances and Clearances

Given the fabrication and erection tolerances of the steel frame, the support details for aluminum curtain walls require field adjustments. The magnitude of the necessary adjustments depends on the number of stories and the span lengths, as discussed in Chapter 4. Suggested adjustments include:

- Means to adjust the slab edge in or out relative to the spandrel beam.
- Means to adjust the location of the curtain wall in or out relative to the slab edge.
- Means to adjust the location of the support connections both vertically and horizontally.

The use of erection bolts in slotted holes with field welding is the most common mechanism for achieving the necessary field adjustments.

The clearance dimensions indicated on the drawings should be equal to the sum of the actual net clearance required plus the outward tolerance for closest element of the primary building frame. The American Architectural Manufacturers Association (AAMA, 1989a) suggests an actual net clearance of not less than 2 in.

9.3.4 Durability

One of the most common problems with curtain wall systems is water leakage when they are not constructed properly. Improperly sealed joints or sealants that do not provide enough allowance for movement can result in water penetration. It is therefore critical that all connections potentially exposed to moisture be adequately protected. Steel anchors are typically galvanized, or stainless steel is used for protection from moisture.

9.3.5 Fire-Safing

Fire-safing is required between the slab edge and the back of the curtain wall frame, when aluminum curtain walls are set so they run by the edge of slab. A clearance between the slab edge and system is required to accommodate erection tolerances, and fire-safing must be used to fill this gap.

The clearance requirements between the slab edge and façade system, with respect to tolerances, adjustment, and fire-safing discussed in Section 8.3.6, also apply to curtain wall systems.

9.4 DESIGN RESPONSIBILITIES FOR ALUMINUM CURTAIN WALLS

As discussed in Chapter 3, it is important that the design team understands the design responsibilities for the support of aluminum curtain walls. Contracts can define the design responsibilities for each project, and one approach commonly found on projects with curtain wall façades is discussed in the following text.

Architect—The architect normally has responsibility for the following:

- Early consultation with a curtain wall consultant or design aid.
- Understanding the influence of different wall types on cost, anchorage strategies, and installation procedures.
- The performance of the curtain wall with respect to moisture and thermal protection of the building, and to the performance of the system including anchorage to the primary building structure.
- The selection of a single- or multi-story span system (usually in consultation with a curtain wall consultant).
- The mullion layout, including spacing and size [usually in consultation with a curtain wall consultant and based on the advice of the structural engineer of record (SER) for practical limits on sizes of spandrels or girts].
- The selection of joint properties and sealant type.
- The selection of the curtain wall thickness and structural performance requirements (usually in consultation with a curtain wall consultant and the SER).
- The plan location with respect to column and spandrel centerlines.
- The selection of the strategy for supporting the curtain wall system (usually in consultation with the SER).
- The setting of acceptable tolerances for erected and adjusted location of the curtain wall.

Structural Engineer of Record (SER)—The SER normally has responsibility for the following:

- The design of the primary building structure, including the slab, slab edge detail, spandrel beam, roll beams, kickers, etc., to support the forces imposed by the curtain wall system with due consideration of stiffness requirements.
- Providing the architect with expected structural design deformations at joints.
- Providing the architect with the vertical and in-plane movements that the curtain wall connections must accommodate for structural deformations, for inclusion in the curtain wall specifications and/or design.
- Providing field adjustable items at the slab edge accommodating some, if not all, of the frame erection tolerance (the amount of adjustability should be coordinated with the requirements in the curtain wall specification).
- Indicating on the structural drawings pertinent assumptions and limitations about the loads from the curtain walls.
- Indicating load support points on the contract drawings.
- Review and approval of shop drawings and field erection drawings for the effect of the aluminum curtain wall and attachments on the primary building structure.

General Contractor—The general contractor normally has responsibility for the following:

- The location of bench marks and offset lines for the curtain wall installer.
- The coordination of the trades involved in installation.

Curtain Wall Manufacturer/SSE—The curtain wall manufacturer and specialty structural engineer (SSE) normally have responsibility for the following:

- The design of the curtain wall frame and its attachments to the primary building structure.

- To provide guidance to the architect in the selection of the appropriate type of curtain wall system and details for the specific building. (This is not typically required of the curtain wall designer, but it is a common occurrence in this industry, as it benefits both the design team and the curtain wall designer as the project develops. The curtain wall manufacturer's experience with various structural systems and installation strategies can be advantageous to the project, especially with early input during the design process.)

- The preparation of shop drawings including details of all attachments to the primary building structure, types and locations of anchors clearly noted, and installation procedures and potential difficulties with field attachment considered and addressed in the shop drawings.

- Clearly marked designation of each piece of the curtain wall system and attachments in accordance with the approved shop drawings, for facilitation of erection.

9.5 CONNECTION TYPES

Proper coordination of the aluminum curtain wall system can prevent costly change orders and problems associated with field installation. In particular, strategies for anchorage to the primary structure should be understood by all parties prior to the curtain wall installation.

Connection types can be divided into two categories: fixed anchors and movable anchors, which are illustrated in Figures 9-5 and 9-6, respectively. Both types are critical to the stability and performance of the curtain wall system, and the effects of each must be understood and incorporated into the design of the supporting building structure. The SER should clearly specify on the contract drawings any limitations on the location and type of each connection.

Details from example projects are shown in Figures 9-7 and 9-8. The SER must detail the supporting primary structure for the localized bending effects that are imposed from curtain wall anchors. Incorporation of additional stiffener plates, heavier sections, and kickers are common methods of accounting for these forces.

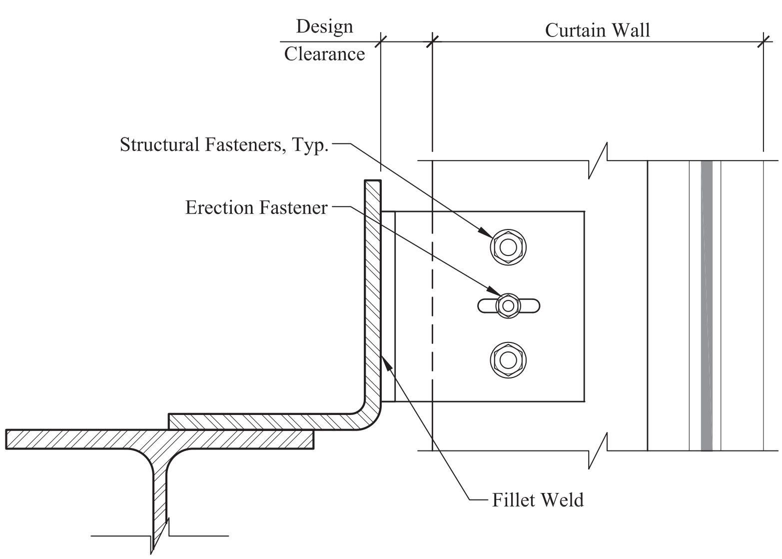


Fig. 9-5. Fixed anchor detail.

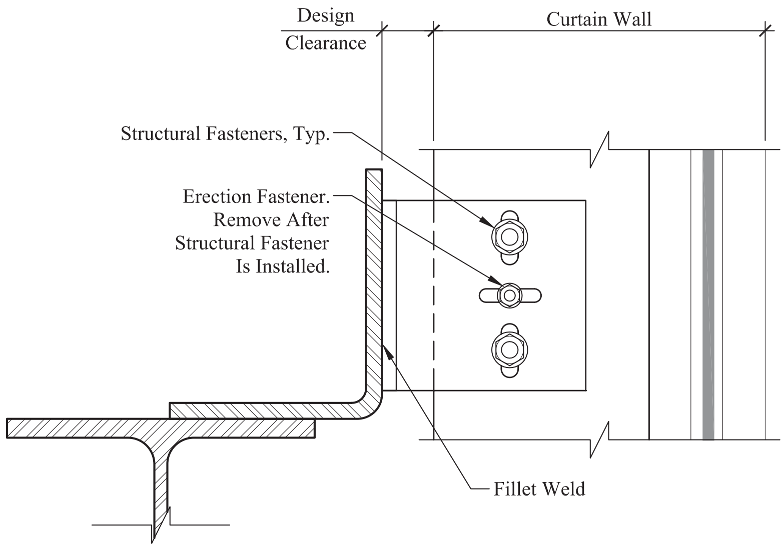


Fig. 9-6. Moveable anchor detail.

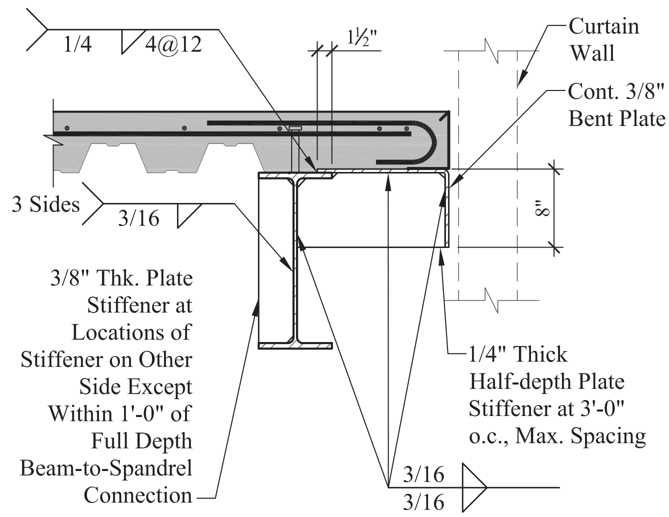


Fig. 9-7. Sample structural steel detail at curtain wall attachment.

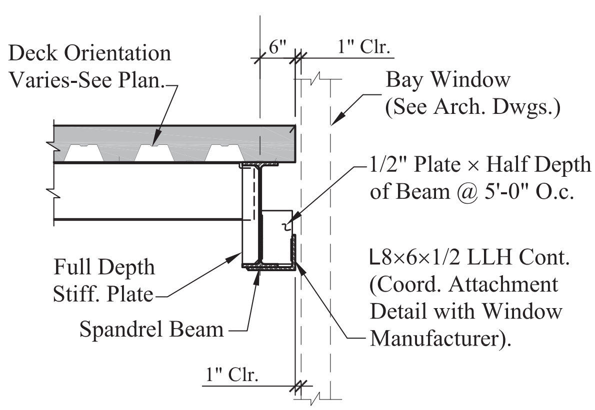


Fig. 9-8. Sample structural steel detail at curtain wall attachment.

Fixed anchors resist forces in any direction, and are commonly referred to as pinned connections. Aluminum curtain walls can exhibit a significant amount of thermal movement, and neglecting to account for such movement could result in buckled mullions and overstressed anchors.

Fixed anchors should also be detailed for field adjustment. This can be accomplished with bolted connections with slotted holes. Once the anchors are adjusted to their proper position, the attachments are welded in place. It should be noted that welding is best limited to steel pieces, as welders require more experience with field welding of aluminum to avoid problems. It is usually more practical to use bolted connections for aluminum pieces.

Movable anchors support the wall system from out-of-plane seismic and wind loads, while allowing movement in the vertical and in-plane directions. Common movable anchor details are illustrated in Figures 9-9 and 9-10. Bolted connections in movable anchors should be tightened such that they allow vertical movement. Washers or slip pads may also be required.

A single-span curtain wall mullion is typically supported by a fixed anchor at one end of its span and a movable anchor at the other. Common methods of providing both types of these attachments at the interface of two vertical mullions are illustrated in Figures 9-11 and 9-12.

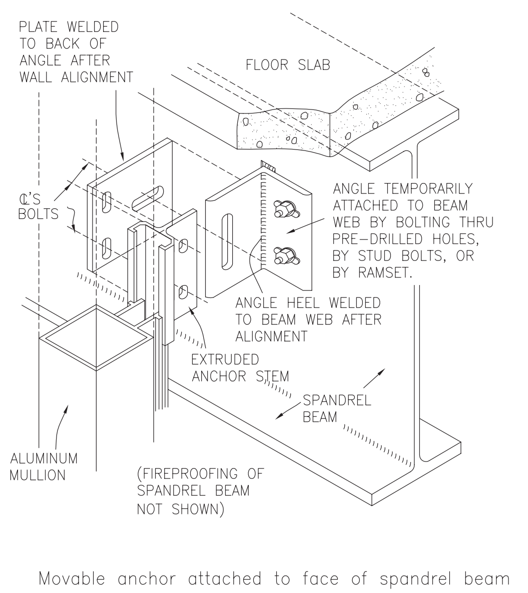


Fig. 9-9. Mullion connection to face of spandrel. (Taken from Curtain Wall Design Manual*, Editorial Revision: May 2005, AAMA. Used with permission, American Architectural Manufacturers Association.)*

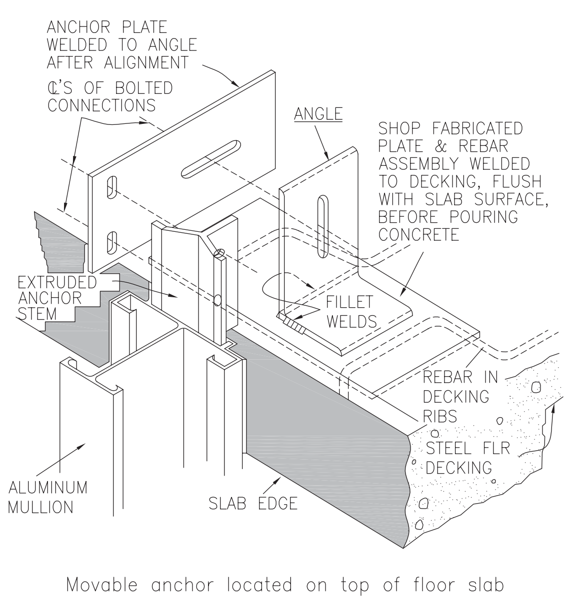


Fig. 9-10. Mullion connection to movable anchor on top of floor slab. (Taken from Curtain Wall Design Manual*, Editorial Revision: May 2005, AAMA. Used with permission, American Architectural Manufacturers Association.)*

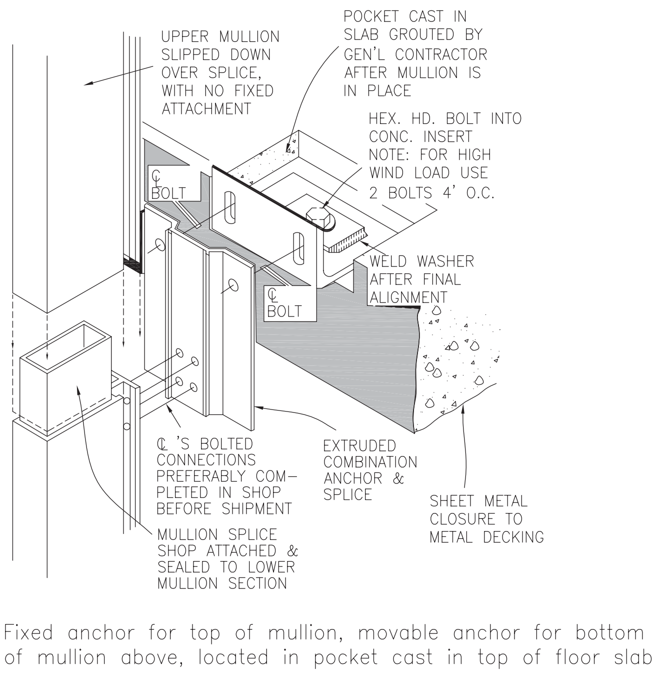


Fig. 9-11. Mullion connection in slab pocket. (Taken from Curtain Wall Design Manual*, Editorial Revision: May 2005, AAMA. Used with permission, American Architectural Manufacturers Association.)*

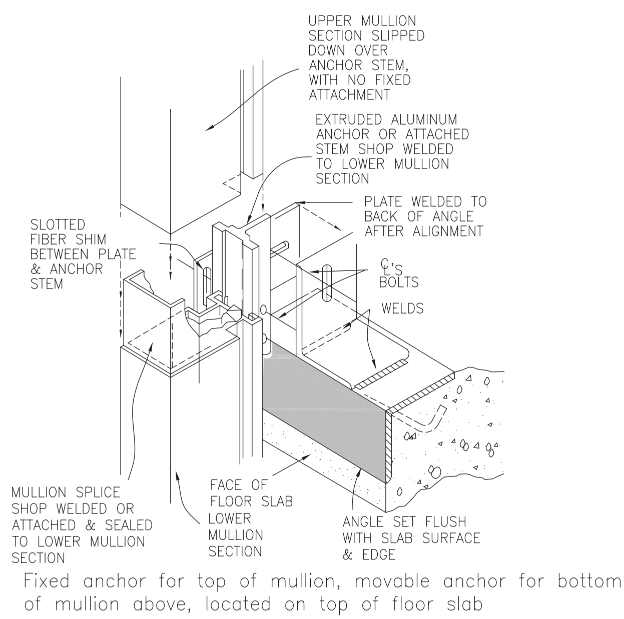


Fig. 9-12. Mullion connection on top of floor slab. (Taken from Curtain Wall Design Manual*, Editorial Revision: May 2005, AAMA. Used with permission, American Architectural Manufacturers Association.)*

There are numerous methods of supporting multi-span curtain walls. Fixed anchors can be located at the top or bottom, or even at the midpoint of the mullion. It is common to provide a fixed anchor at the bottom of the first-story mullion and fixed anchors at the midpoints of two-span mullions above the first floor.

One of the most common methods of anchoring curtain wall systems between floors is with the use of receptor channels, particularly in horizontal strip window anchorage. Receptors of extruded aluminum can be used at the head of the curtain wall, sill, and at the jambs. They are capable of providing allowance for movement of the primary building structure while being incorporated into the drainage and thermal barrier systems.

Curtain wall attachment pieces can be either steel or aluminum. Steel anchors are often utilized because of their greater strength, weldability, and more ready availability (aluminum anchors are more typically custom made). Aluminum anchors can be more versatile in the wall design because of their custom shapes, but they also tend to be more costly. The degree of exposure of the anchors to moisture can also suggest the more suitable material. In exposed conditions, aluminum anchors are often selected over steel, because steel anchors typically require a protective coating to prevent corrosion. Consideration must also be given to connections between steel and aluminum parts. Without proper separation at meeting surfaces, moisture can lead to a galvanic reaction, resulting in corrosion. Plastic pads or Teflon® washers are commonly used to isolate the materials.

9.6 POTENTIAL PROBLEMS WITH SUPPORT AND ANCHORAGE OF ALUMINUM CURTAIN WALLS

The following is a list of potential problems that designers should be aware of, and avoid, when designing support and anchorage of aluminum curtain walls. There is no particular meaning implied by the order of the list.

- Large gaps between the anchors and the primary building structure can result in excessive bending effects. Welded shims are commonly used to fill and reduce the potential for overstressed anchors.

- Lack of coordination of locations of adjustment. If vertical and horizontal adjustments are to be made solely in the attachments of the curtain wall system to the primary building structure, slotted holes must be long enough to account for all of the required adjustment. If the curtain wall designer is relying on adjustments being made through the primary building structure rather than the façade attachments, the slotted holes provided may be inadequate in length.

- Lack of coordination of bolted attachments to the primary building structure. Locations of bolt holes for curtain wall attachment should be coordinated with the steel fabricator so that holes can be made in the shop.

- Inadequate detailing in mullion splices (which are the responsibility of the SSE) for volume changes and movement of the primary building structure.

Chapter 10
Glass-Fiber-Reinforced Concrete (GFRC) Panels and Other Lightweight Systems

10.1 GENERAL DESCRIPTION OF GLASS-FIBER-REINFORCED CONCRETE PANELS AND OTHER LIGHTWEIGHT SYSTEMS

Glass-fiber-reinforced concrete (GFRC) panels are a versatile and lightweight façade system. Various surface colors and textures are available, making GFRC adaptable to numerous architectural applications. They can be designed with a face mix to achieve an appearance very similar to that of precast concrete panels.

GFRC and other lightweight systems, such as thin brick and stone facings, are beneficial because they are easy to erect and generally do not require heavy steel structural support in the primary building structure. For these reasons, GFRC and other lightweight systems often result in more economical steel frames.

GFRC panel systems typically weigh between 9 and 25 psf. Actual weights depend on the type of surface finish, panel shapes and sizes, and back-up frame composition. Typical wall systems are comprised of the panel skin, anchors attaching the skin to the back-up frame, the panel back-up frame, and connectors attaching the panel frame to the primary building structure. Insulation is often located within the panel back-up frame. The panel skin is made up of cement/aggregate slurry and alkali-resistant glass fibers. Skins are typically sprayed-applied onto forms, as opposed to the less common premixing method. The spraying process involves the simultaneous spraying of glass fibers and slurry onto a mold, and subsequent compaction of the material. Premixing is the process in which the cement, sand, chopped glass fiber, and water are mixed together into mortar and then sprayed or cast into a product (PCI, 2001).

Wall systems are primarily either single-skin GFRC panels or sandwich panels. Sandwich panels are panels that have a lightweight insulating core with a panel skin attached on each side. This chapter focuses on the more commonly used single-skin panels.

GFRC panels are not load bearing and are not considered part of the lateral load resisting system. The primary building structure must be designed with its own lateral force resisting system and must support the weight and lateral loads from the panel system. Out-of-plane lateral loads and in-plane seismic loads from their own self-weight are transferred from the panel skin to the primary building structure. The GFRC skin and the back-up frame comprise the panel. For a common panel system made with a cold formed metal back-up frame, the load path from the panel skin is through the skin anchors to the metal studs, from the studs to the upper and lower stud tracks, from the tracks to the connector studs, and from the connector studs to the connector elements that attach to the primary building structure. These system components are illustrated in Figures 10-1 and 10-2.

GFRC panels comprise a barrier wall system for water management. As with all barrier wall systems made from panels, the joints are the key to successful moisture protection performance of the wall system. The joint system should include watertight sealants and should be designed to handle the anticipated movement at the joints.

GFRC and other lightweight panels can either be in-filled between floors or run continuously past the outside face of the floor slab edge. Continuous systems outside of the floor slab edge are common for multi-story panels and spandrel panels supporting horizontal strip windows. GFRC panels commonly have been manufactured up to 30 ft in length.

10.2 STRATEGIES FOR SUPPORT OF GFRC PANEL SYSTEMS

Strategies for support of GFRC façades include anchoring the GFRC skin to the back-up frame and supporting the back-up frame from the building structure. Initial cracking problems with GFRC panels have to do with anchoring the GFRC skin to the back-up frame in a manner that restrains the movement of the skin. The main concern designers have when choosing the strategy to support the back-up frame on the structure is ensuring sufficient allowance for relative movement between the back-up frame and the structural frame.

The attachment of the panel skin to the back-up stud frame is commonly accomplished in one of two methods. In both methods, all anchors transfer out-of-plane lateral loads to the back-up frame. The most common method includes a combination of flex anchors and seismic and gravity anchors. Figure 10-3 illustrates this strategy for GFRC panel system design. In this strategy, seismic anchors transfer in-plane loads to the back-up frame, gravity anchors support the skin weight, and flex anchors provide only out-of-plane wind and seismic support while allowing in-plane lateral movement. Gravity connections can be achieved by bonding an anchor to the skin through the use of a bonding pad, and attaching the anchor to the back-up stud frame with a welded connection.

Alternatively for lightweight panel systems, the skin can be attached to the back-up frame with flex anchors only. In this

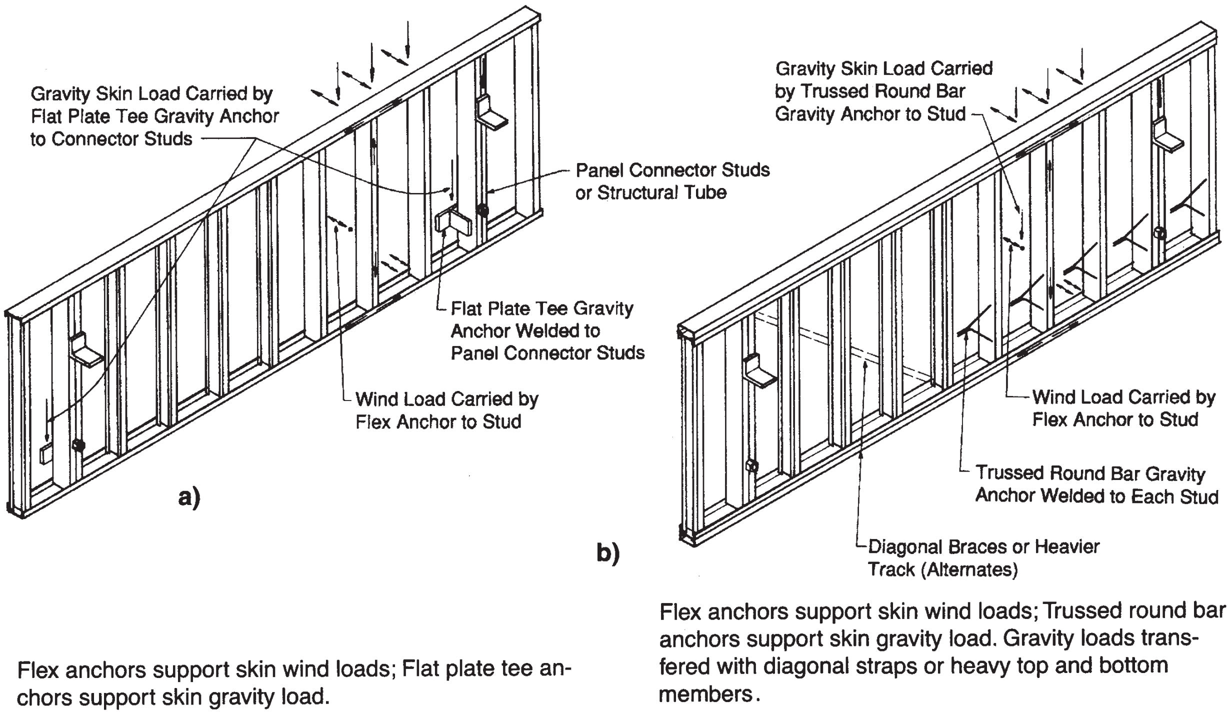


Fig. 10-1. Gravity anchor and panel framing systems. (Taken from Glass Fiber Reinforced Concrete Panels*, Fourth Edition, PCI. Used with permission, Precast/Prestressed Concrete Institute.)*

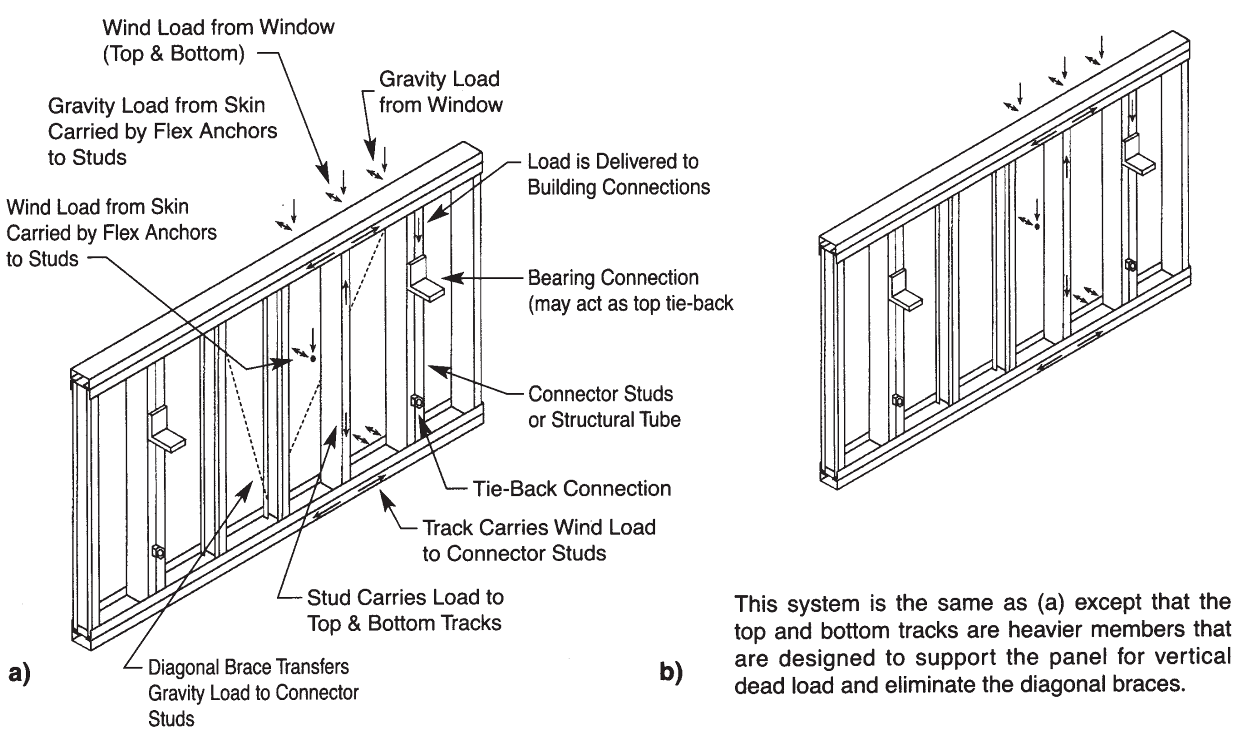


Fig. 10-2. Optional methods of stiffening frame for panels without gravity anchors—for small panels only. (Taken from Glass Fiber Reinforced Concrete Panels*, Fourth Edition, PCI. Used with permission, Precast/Prestressed Concrete Institute.)*

case, the flex anchors transfer gravity, wind, and seismic loads to the frame from the panel skin, but must be flexible enough that they do not restrain in-plane volume changes in the skin.

GFRC panel back-up frames are typically attached to the primary building structure with concepts similar to precast concrete panels. A combination of two bearing connections and at least two tie-back connections per panel are used. As with precast concrete panels, it is good practice to not use more than two bearing connections per panel frame, as the reactions become statically indeterminate. However, with GFRC panels, the use of more than two tie-back connections per panel frame is common without great concern for restraining thermal bowing in the panel.

The erection process for GFRC panels is also similar to precast concrete panels. As discussed in Chapter 8, adequate adjustability must be provided in the vertical direction and in both horizontal directions for all bearing and tie-back connections. Attachments to the primary building structure can be either bolted or welded. If fireproofing is required, it is preferred that fireproofing be applied after panel erection, adjustment, and final attachment. Otherwise, block-outs should be provided and patched after panel attachment in order to facilitate panel installation.

10.3 PARAMETERS AFFECTING THE DESIGN OF GFRC PANEL SUPPORTS

Parameters affecting the design of GFRC panel supports include:

- Architectural decisions.
- Movement requirements.
- Field adjustability for tolerances and clearances.
- Durability.
- Fire-safing.

10.3.1 Architectural Decisions

Architectural decisions can dictate the strategy for supporting the GFRC panel system. The lightweight nature of the system makes GFRC panels similar to various curtain wall systems discussed in this Design Guide, yet the methods of attachment are similar to those used for precast concrete panel systems. The number of factors to consider in the design of GFRC panels makes the support system one of the more complex systems to detail architecturally. Considerations include aesthetics, location of joints, joint width, panel configuration, cavity width, story height, erection procedures, and economics. Panel configuration has the most influence on attachment of the back-up frame to the structure.

Aesthetics—Wall panel systems are primarily comprised of either full-story-height panels, or bands of horizontal spandrels. Continuous horizontal bands, or very wide bands, of windows require support at both the head and sill of the windows. Spandrel GFRC panels typically provide lateral support at the head of the window and gravity and lateral support at the sill of the window. Figure 10-3 illustrates the conceptual difference between full-story panel frame support and spandrel-panel frame support. Locations of windows must also relate to panel type and layout. Windows supported by more than one panel can result in joint sealant failure from differential movement of the panels.

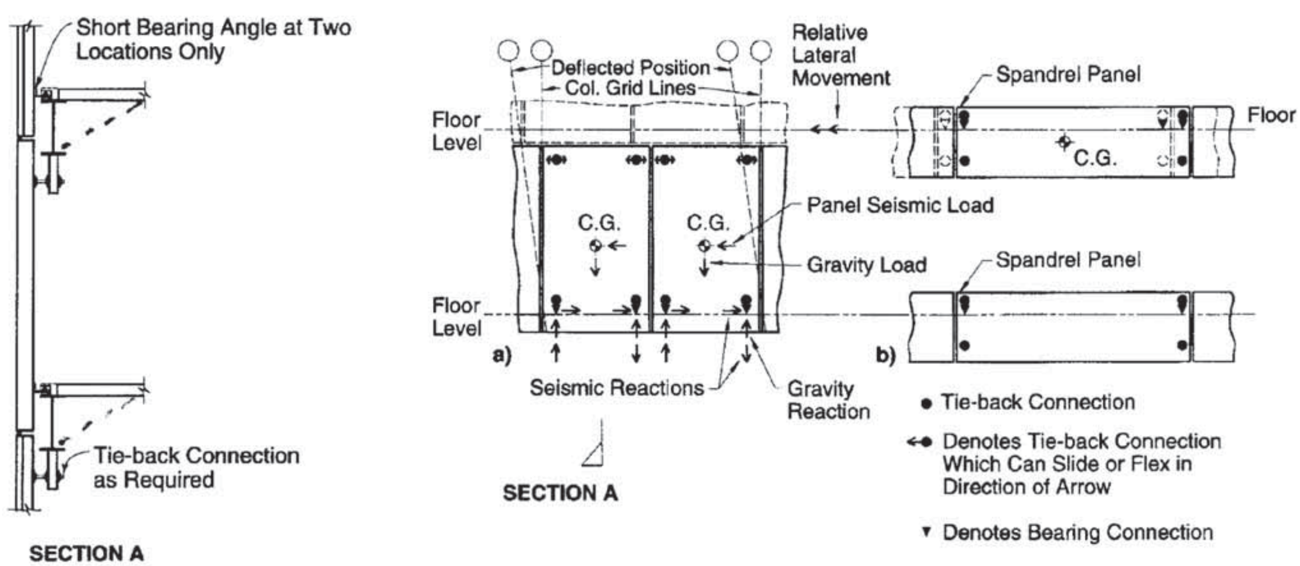


Fig. 10-3. Panel connection concepts. (Taken from Glass Fiber Reinforced Concrete Panels*, Fourth Edition, PCI. Used with permission, Precast/Prestressed Concrete Institute.)*

Joints and panel configuration—Joint sizes and locations are directly related to both the support and performance of the GRFC panel system. Good joint design is based on panel size, tolerances, anticipated movement of the primary building structure and the panel system, and joint materials. Panel configuration must be established both in elevation and in plan, with respect to the slab edge and supporting structure. Panel back-up frames can either be in-filled between floors or run continuously past the outside face of the floor slab edge.

It is usually economical if the horizontal joints are located within the depth of the spandrel beam, so that the GFRC panel frame can be attached to the supporting structure without the use of hangers. The closer the anchors are to the spandrel beams, the less distance the supporting attachment steel needs to extend, and this generally means less cost.

Cavity width—As further discussed in the "Durability" section, common designs include an air space cavity between the panel skin and the insulation to allow air flow and prevent the build-up of moisture in the system. The width of the cavity affects the length of the connections between the panel skin and the back-up frame. Fire-safing methods are also dependent on the cavity width and plan configuration of the panels. See Figure 10-4 for common fire-safing details.

Story height—The story height directly impacts the weight and thickness of the system. The story height also impacts the magnitude of the out-of-plane forces that must be transferred from the back-up frame to the primary structure.

Erection procedures—As with precast concrete panels, erection equipment often influences GFRC panel size and should be considered in the selection of panel layout.

Economics—Panel sizes and shapes may be selected to minimize handling and erection time. Panels having similar connections increase overall efficiency. The workers' familiarity with the connections results in increased productivity. Panel layouts that are repetitive in finish, shape, and size will often prove more economical.

10.3.2 Movement Requirements

Both the GFRC skin and building frame movements must be accounted for in the design of GFRC panel joints. The design of the skin–to–back-up anchors must account for the skin movements. The back-up–to–structure attachments must account for the building frame movements.

Movement can be an effect of combinations of temperature, shrinkage, creep, moisture, vertical and lateral loads, and deflection of the building frame members. GFRC panel systems can be more sensitive to excessive restraint than other systems, and sometimes panels crack as a result of excessive restraint.

Undesired restraint can result from such conditions as overtightened bolted connections and/or the attachments of window frames or signs to the panels. Teflon® or nylon washers are a common method of providing necessary allowance for movement at bolted connections.

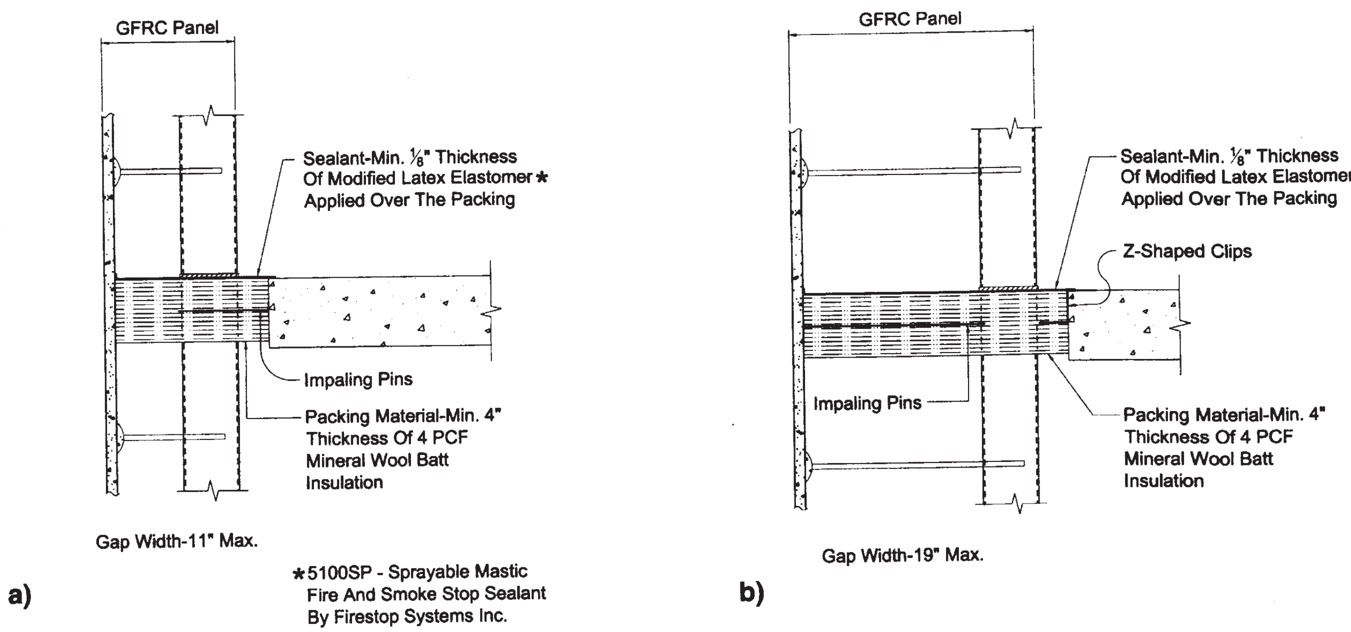


Fig. 10-4. Fire-safing details. (Taken from Glass Fiber Reinforced Concrete Panels*, Fourth Edition, PCI. Used with permission, Precast/Prestressed Concrete Institute.)*

GFRC panels are also perhaps affected more significantly by a broader range of movement sources than most other façade systems. Shrinkage in particular can be significant because of the high percentage of cement in GFRC. Because GFRC panels are typically thinner than precast concrete panels, they also tend to experience greater changes in temperature and moisture.

The sum of the movement accommodated by the soft joint will be approximately 25 to 50 percent of the soft joint size, depending on sealant materials and sealant bond. Panel joint width should be a minimum of ¾ in.

The skin must be anchored to the back-up frame with enough support stiffness to mitigate cracking of the panel due to support deflection. Furthermore, the structural frame must have sufficient stiffness to adequately support the panel back-up frame. Limiting the spandrel beam and associated support deflections to less than $L/240$ for out-of-plane lateral loads, where L is the span, is usually sufficient for most GFRC panels. There is no industry standard of practice for allowable vertical movement due to support deflection as there is with masonry cavity wall systems because there are any number of GFRC panel and back-up frame configurations, each with its own limitations. The building frame stiffness requirements are usually governed by that needed for acceptable panel joint designs.

10.3.3 Field Adjustability for Tolerances and Clearances

GFRC panel systems require field adjustments for fabrication and erection tolerances of the steel frame. The magnitude of the necessary adjustments depends on the number of stories and the span lengths as discussed in Chapter 4. Suggested adjustments include:

- Means to adjust the slab edge closure plate in or out relative to the spandrel beam.
- Means to adjust the location of the GFRC panel frames in or out relative to the slab edge.
- Means to adjust the location of the GFRC panel frames up or down relative to the slab edge.
- Means to adjust the location of attachment anchors both vertically and horizontally.

Hardware with slotted holes and erection bolts allow horizontal adjustment, and shim plates are often used to provide vertical adjustment. If shims are used, they must be placed underneath the panel back-up frames, never underneath the GFRC skin. Placement under the skin could cause unanticipated loads on the GFRC skin, resulting in cracking and potential failure. For tie-back connections, it is common for designs to use a long, threaded rod, which allows lateral movement in all directions with little restraint from the flexibility of the rod in bending.

Tolerances relating to production and erection are discussed in detail in Chapter 4, and tolerances in combination with clearances must be considered. Common clearances and cover dimensions considering panel performance as well as production and erection procedures include the following (PCI, 2001):

- Panel skin to back-up frame: ½ in. min.
- GFRC backing to flex anchors: ⅛ in. to ⅜ in.
- Bonding pad thickness over top of flex anchor: ½ in. min.
- Between back of panel frame and fireproofing on steel member: 1½ in. min.
- Between back of panel frame and steel member (if not fireproofed): 1½ in. min.
- Exterior face of panel skin to exterior face of insulation (if used): 3 in. min.

10.3.4 Durability

Current design methods for GFRC panels incorporate an air space between the panel skin and the insulation to mitigate moisture accumulation. Vapor barriers are often used as an additional protective measure. Common practice is to place the vapor barrier on the interior side of the insulation in cold-weather environments, and on the exterior of the insulation in warm-weather environments. Any steel on the exterior side of the vapor barrier should be given protection from moisture build-up, which could lead to corrosion. Common protective methods are galvanizing, cadmium plating, and zinc plating. The stud back-up frame is typically coated with G-60 galvanizing. Hot-dip galvanizing or zinc-rich paint is common for attachment hardware.

10.3.5 Fire-Safing

As discussed earlier in this chapter for other systems, fire-safing is sometimes required to fill the gap between the floor slab edge and the inside face of GFRC skin. Mineral wool batt insulation commonly is used to pack the gap, and a fire and smoke sealant is applied on the top surface of the insulation.

It is generally cost effective to have an adjustable slab edge detail to accommodate at least some of the frame tolerances to avoid excessive fire-safing.

10.4 DESIGN RESPONSIBILITIES FOR GFRC PANEL SYSTEMS

As discussed in Chapter 3, it is important that the design team understands the design responsibilities for the support of GFRC panel systems. Contracts can define the design responsibilities for each project, and one approach commonly

found on projects with GFRC panels is discussed in the following text.

Architect—The architect normally has responsibility for the following:

- The GFRC panel finish, shape and size.
- The performance of the GFRC panel system with respect to moisture and thermal protection of the building.
- The selection of the GFRC panel system type (single-skin or sandwich panels, full-story or spandrel panels).
- Determination of the required fire resistance rating.
- The location of horizontal soft joints (and thus influencing the system of support framing).
- The dimension of the horizontal soft joint and performance of sealant joints (usually in consultation with the SER to understand practical limits for controlling movement from the steel frame and support assembly).
- The GFRC panel plan location with respect to column and spandrel centerlines.
- The setting of acceptable tolerances for erected and adjusted location of panels.
- The test method to be used for performance specifications of the panel system.

Structural Engineer of Record (SER)—The SER normally has responsibility for the following:

- The design of the primary building structure, including the slab, slab edge detail, spandrel beam, roll beams, hangers, kickers, etc., to support the forces imposed by the GFRC panel system with due consideration of stiffness requirements.
- Providing field adjustable items at the slab edge to accommodate some, if not all, of the frame erection tolerance (the amount of adjustability should be coordinated with the requirements in the GFRC specification, which is especially important if the panel back-up frame is infilled between floor slabs).
- Indicating on the structural drawings pertinent assumptions and limitations about the loads from the panels.
- Indicating load support points on the contract drawings.
- Providing the architect with the vertical and in-plane movements that the GFRC panel frame connections must accommodate for structural deformations, for inclusion in the GFRC specifications and/or design.
- Providing the architect with expected structural design deformations at joints.
- Review and approval of shop drawings and field erection drawings for the effect of the GFRC panels and attachments on the primary building structure.

General Contractor—The general contractor normally has responsibility for setting the benchmarks to which the GFRC panel installer refers when aligning his work.

GFRC Panel Manufacturer and SSE—The GFRC panel manufacturer and specialty structural engineer (SSE) normally have responsibility for the following:

- The design of the GFRC skin, attachments of the skin to the back-up frame, the back-up frame itself, and the connectors that transfer loads from the GFRC panel frame to the primary building structure.
- To clearly mark each panel to correspond with the designations on the approved shop drawings.
- To provide testing of anchors to establish adequate safety factors.
- For loose connection hardware and devices for lifting and erection.

10.5 CONNECTION TYPES

The back-up frame of the GFRC panel is attached to the primary building structure with bearing and tie-back connections similar to those used with precast concrete panels. Refer to Chapter 8 for detailed descriptions of bearing and tie-back connection concepts. The most significant difference between precast concrete panel connections and GFRC panel connections, however, is the demand on the anchors and connectors. GFRC panels are much lighter than precast concrete panels, so attachments can be smaller and less costly.

Bearing connections—These can often be lightweight material such as structural steel angles or bent plates. Stiffener plates can be provided, if necessary, to prevent rotation of the vertical leg of the angle/bent plate.

Given the relatively light weight of the panels, they can usually bear on the slab edge or be attached to a bent plate anchored to the slab.

Tie-Back Connections—These are typically made with threaded rods, but can be metal straps in lightly loaded conditions.

Figures 10-5 and 10-6 illustrate typical bearing and tie-back connections.

Alignment Connections—These are typically used to adjust the position of individual panel frames relative to each other. A typical method of alignment between adjacent frames is illustrated in Figure 10-7.

Typically the responsibility of the SSE, flex anchors and gravity anchors are used to attach the GFRC skin to the panel back-up frame. Anchors can be made with bent plates or trussed rods as illustrated in Figures 10-8, 10-9, and 10-10.

10.6 GFRC SINGLE-SKIN FULL-STORY PANEL

GFRC panels are typically spandrel-supported rather than column-supported due to practical limitations in panel length. Single-skin full-story panels typically bear on the slab edge or directly on the spandrel steel at the floor level near the bottom of the panel, and have tie-back connections located near the top of the panel at the floor level above the bearing floor. A typical method of support is illustrated in Figure 10-11. Loads resulting from the eccentricity of full-story hung GFRC panel systems typically have minimal impact on the design of the primary building structure. GFRC panels are lightweight, and story-tall panels spread the overturning eccentricity, resulting in easily managed stabilizing reactions.

10.7 GFRC SINGLE-SKIN SPANDREL PANEL

Single-skin spandrel panels are often utilized in architectural layouts with continuous horizontal bands of windows. Eccentricities in spandrel-supported spandrel panels are often stabilized with the use of hangers and kickers. Although the stabilizing forces are not typically as large as with precast concrete panels, there remains the same need to coordinate kickers and hangers with ceiling height and mechanical systems. Figure 10-12 illustrates a typical method of support for spandrel panels.

Spandrel-supported spandrel panels typically have two connections located near the top of the panel that bear on the structure at the floor level. It is common to bear on the slab edge or steel brackets. Tie-back connections are often located near the bottom of the panel, and attach either to the bottom flange of the spandrel beam or to a hanger beneath the spandrel beam. Additional tie-back connections are sometimes provided for more out-of-plane lateral load resistance.

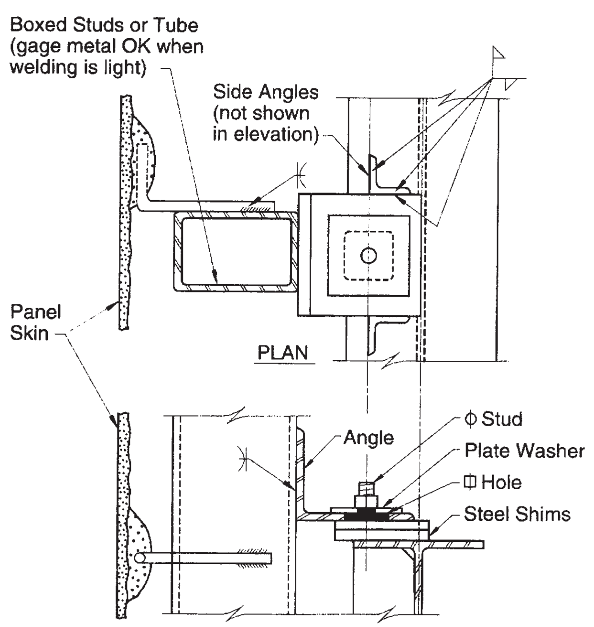


Fig. 10-5. Bearing connections—combined with tie-back.
(Taken from Glass Fiber Reinforced Concrete Panels*,*
Fourth Edition, PCI.
Used with permission, Precast/Prestressed Concrete Institute.)

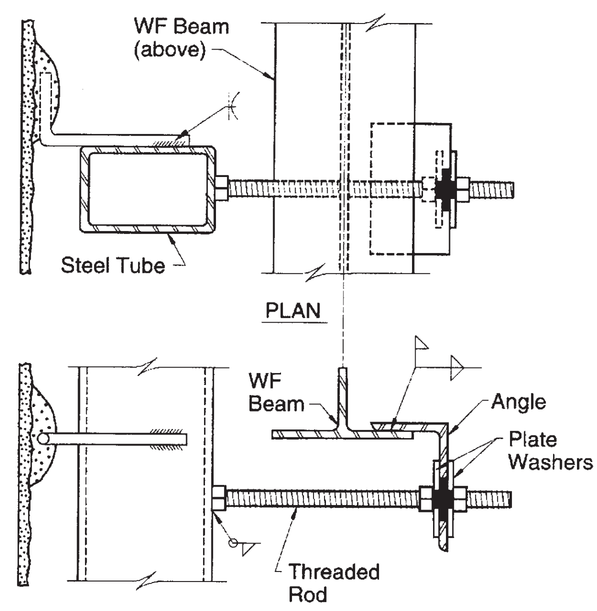


Fig. 10-6. Tie-back connections.
(Taken from Glass Fiber Reinforced Concrete Panels*,*
Fourth Edition, PCI.
Used with permission, Precast/Prestressed Concrete Institute.)

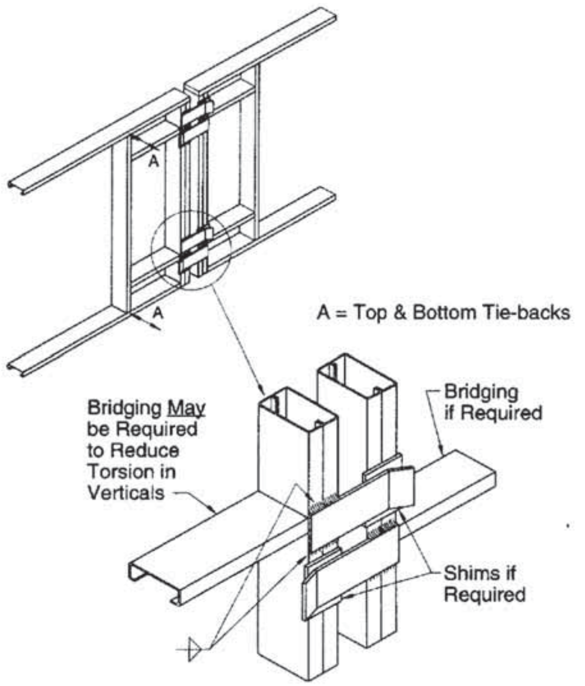


Fig. 10-7. Alignment connections. (Taken from Glass Fiber Reinforced Concrete Panels*, Fourth Edition, PCI. Used with permission, Precast/Prestressed Concrete Institute.)*

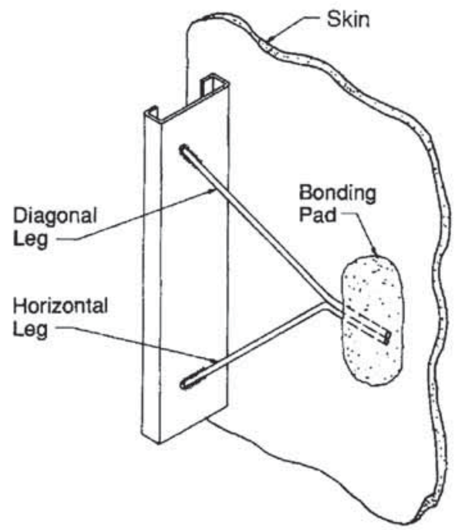


Fig. 10-8. Trussed rod gravity anchor. (Taken from Glass Fiber Reinforced Concrete Panels*, Fourth Edition, PCI. Used with permission, Precast/Prestressed Concrete Institute.)*

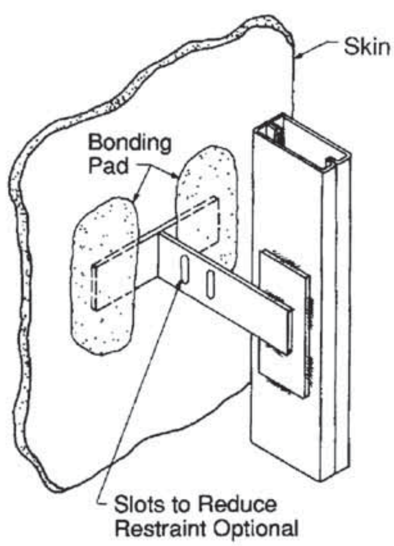


Fig. 10-9. Plate gravity anchor. (Taken from Glass Fiber Reinforced Concrete Panels*, Fourth Edition, PCI. Used with permission, Precast/Prestressed Concrete Institute.)*

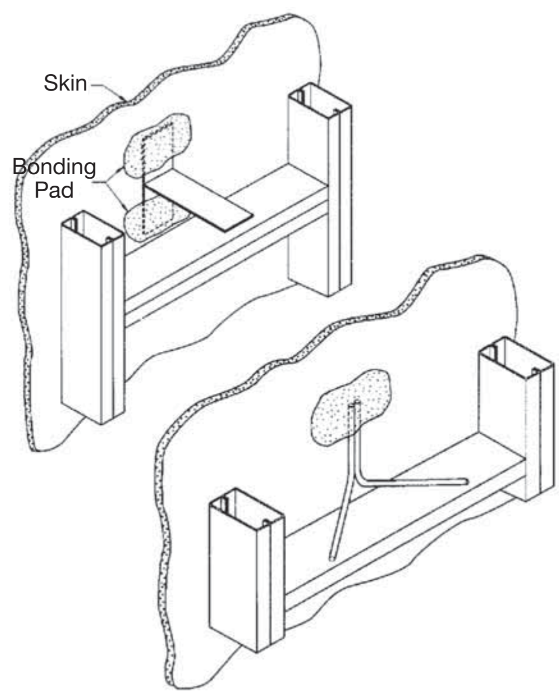
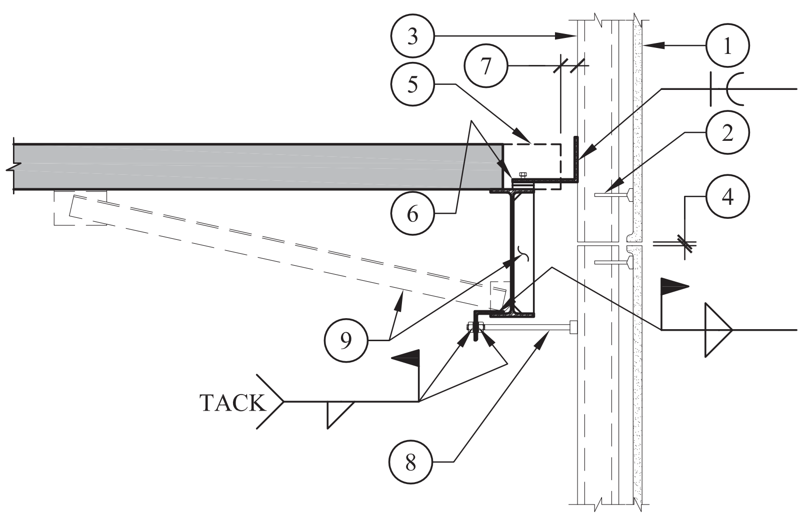


Fig. 10-10. Plate and trussed seismic anchors. (Taken from Glass Fiber Reinforced Concrete Panels*, Fourth Edition, PCI. Used with permission, Precast/Prestressed Concrete Institute.)*

10.8 POTENTIAL PROBLEMS WITH SUPPORT AND ANCHORAGE OF GFRC PANEL SYSTEMS

The following is a list of potential problems that designers should be aware of, and avoid, when designing support and anchorage of masonry cavity walls. There is no particular meaning implied by the order of the list.

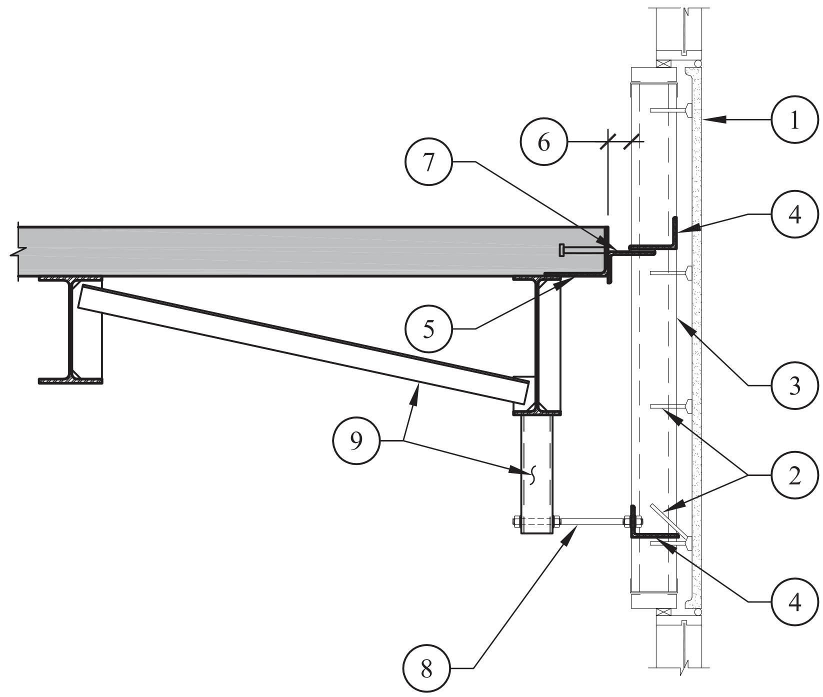
- The skins of panels with two material skins should have similar volume change characteristics in order to avoid excessive panel bowing.
- Bowing or warping as a result of cantilevered skins at panel edges.
- Corrosion of support elements and attachments as a result of leaking air and water.
- Insufficient coordination with GFRC manufacturer and SSE.
- Over-restraint of panel movement due to thermal and moisture cycles.
- Inadequate allowance for movement of the building frame.



NOTES:

1. GFRC panel skin.
2. Skin anchors to backup frame.
3. Backup frame. Connector studs at attachment locations.
4. Panel joint.
5. Slab box out for bearing attachment. Box outs allow attachment to be hidden when the pocket is filled. Alternatively, the attachment can be to top of slab without box outs.
6. Bearing attachment to provide both vertical and horizontal adjustment. Eccentricity between panel C.G. and center of bearing is resolved by horizontal forces in attachments at top and bottom of panel and bending of attachment cantilevering from connector stud.
7. Slab edge clearance is 1 in. minimum, 2 in. preferred. Provide adjustable slab edge detail, if possible.
8. Tie-back attachment. Lateral flexibility through bending of threaded rod.
9. Stiffener and kicker if needed for eccentric tie-back force at bottom flange.

Fig. 10-11. Full-story panel connection.



NOTES:

1. GFRC panel skin.
2. Skin anchors to backup frame.
3. Backup frame.
4. Connector angles in backup frame.
5. Bent plate at slab edge designed as pour stop and anchored to slab to transfer panel loads to the slab.
6. Slab edge clearance is 1 in. minimum, 2 in. preferred. Provide adjustable slab edge detail, if possible.
7. Attachment clip. Attachment to provide both vertical and horizontal adjustment.
8. Tie-back attachment. Lateral flexibility through bending of threaded rod.
9. Hanger and kickers as required for tie-back forces.

Fig. 10-12. Strip window panel connection.

Chapter 11
Exterior Insulation and Finish System (EIFS) Panels

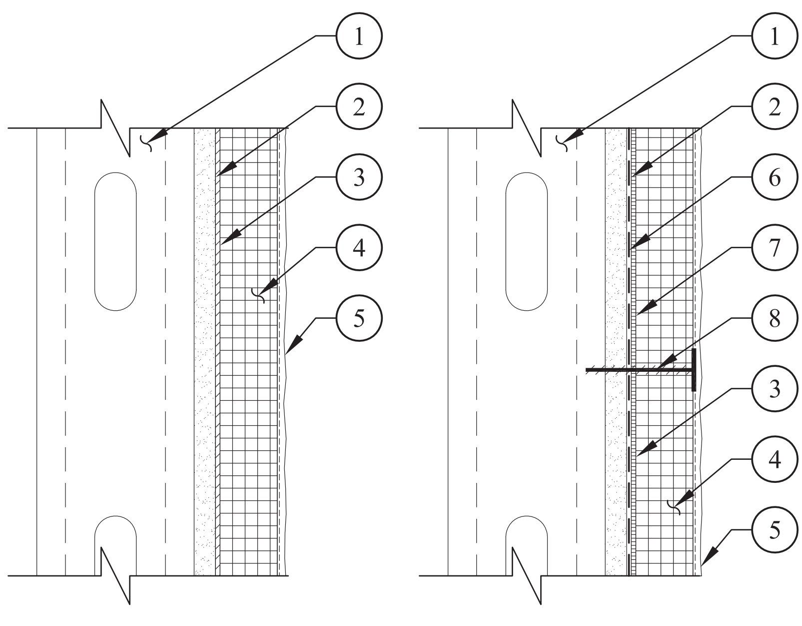
11.1 GENERAL DESCRIPTION OF EXTERIOR INSULATION AND FINISH SYSTEM (EIFS) PANELS

Compared to the other façade systems discussed in this Guide, exterior insulation and finish system (EIFS) panels are a relatively new cladding system with few standards. EIFS can either be fabricated in panels or site-applied. Site-applied EIFS is often more cost effective, but it is not typically erected during cold weather and is not feasible for short construction periods and buildings that do not have scaffolding access for installation. This Design Guide focuses on EIFS panels rather than site-applied EIFS.

EIFS panels are comprised of EIFS and a substrate system. EIFS consists of (from exterior face inward) lamina (finish and base coat), expanded polystyrene insulation, and adhesive (or mechanical or metal lath fastener) attachment to the substrate. The substrate consists of (from the EIFS inward) exterior sheathing, metal studs with bridging, interior wall board, and the interior finish. The exterior sheathing is typically attached to the metal studs with self-drilling screws.

Since their first use, EIFS panels have been a barrier-type cladding. Frequent problems with moisture penetration and subsequent deterioration of substrates have led some to revise commonly used details. Modern concepts of EIFS incorporate a drainage plain. Figure 11-1 illustrates conventional and drainable EIFS.

EIFS panels are custom designed for specific project applications and offer a tremendous amount of flexibility to the cladding design. These very lightweight panels can be spandrel, full-story, in-filled between floor slabs, multi-story, or even three-dimensional panels. Their appearance can easily be made to resemble precast concrete panels.



(a) Conventional EIFS

(b) Drainable EIFS

NOTES:

1. Metal stud backup panel frame.
2. Sheathing.
3. Adhesive.
4. Rigid expanded polystyrene insulation board (EPS) insulation.
5. Polymer-modified cement base coat, glass-fiber fabric, and acrylic-based, textured finish.
6. #15 asphalt-saturated felt paper.
7. Ribbed PVC lathe for drainage plane.
8. Mechanical fastener with disc washer.

Fig. 11-1. Conventional vs. drainable EIFS.

EIFS panels are not typically load bearing and are not considered part of a lateral load resisting system. The primary building structure must be designed with its own lateral force resisting system and must support the weight and lateral loads from the EIFS panels. Out-of-plane lateral loads and in-plane seismic loads from the panels are transferred from the EIFS through the sheathing fasteners, into the metal stud substrate, and through the connector attachments to the primary building structure.

11.2 STRATEGIES FOR SUPPORT OF EIFS PANEL SYSTEMS

EIFS panel frames are typically attached to the primary building structure through the use of fixed and movable connections with concepts similar to the attachment of aluminum curtain wall systems. As with other hung façade systems, it is not recommended to use more than two bearing connections per panel, as the reactions would become statically indeterminate. Figures 11-2 through 11-4 illustrate common methods of EIFS panel attachment at floor slabs and spandrel beams. EIFS panels that are continuous past the outside face of the floor slab are common for full-story, multi-story, and spandrel panel strategies.

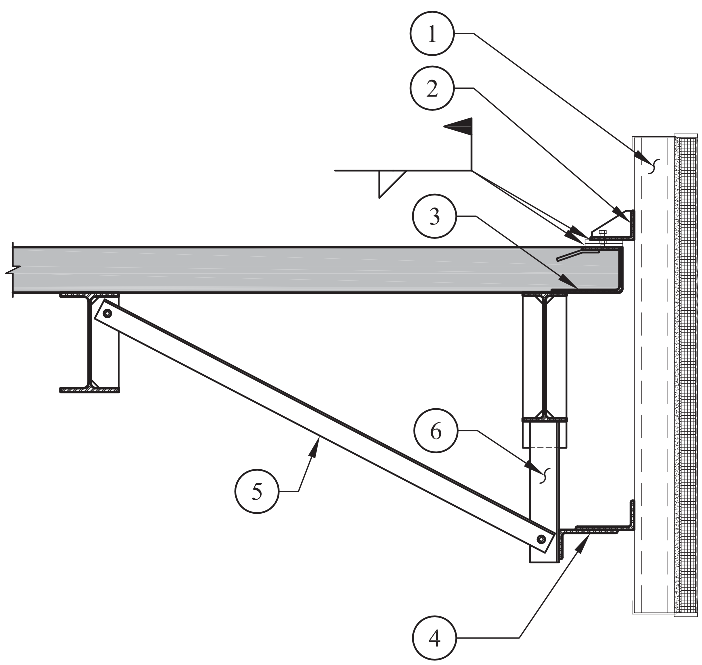
Figure 11-2 demonstrates a typical method of spandrel panel support used with continuous horizontal window bands. Full-story panels can be supported in a manner as shown conceptually in Figure 11-3. A panel joint will exist between the gravity support attachment at the top of the slab and the tie-back connection at the bottom of the spandrel as illustrated. When panels are multi-story, the attachments at intermediate floors are only lateral attachments for out-of-plane support of the panel. Figure 11-4 shows an example of an in-fill panel system.

Panel back-up frames for commercial use are commonly made of steel. Often the back-up frames are comprised of light-gage metal studs with bridging, but structural steel members are sometimes used in conditions of high forces in the frame. Back-up frames are typically anchored to the building with welded connectors.

11.3 PARAMETERS AFFECTING THE DESIGN OF EIFS PANEL SUPPORTS

Parameters affecting the design of EIFS panel supports include:

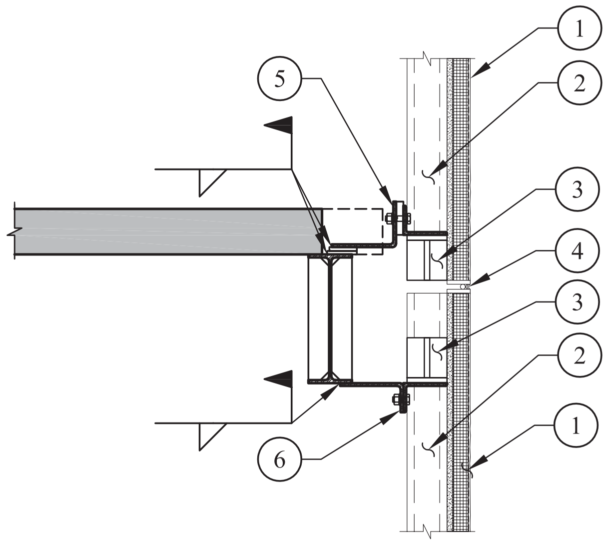
- Architectural decisions.
- Movement requirements.
- Field adjustability for tolerances and clearances.
- Durability.
- Fire-safing.



NOTES:

1. EIFS spandrel panel with backup frame.
2. Eccentric bearing attachment. Provide sufficient overlap for in-out adjustment. Shims and leveling bolts for vertical adjustment.
3. Bent-plate pour stop. Provide in-out adjustment to minimize fire-safing and overlap of bearing connection.
4. Tie-back connection. Vertical and horizontal movement not necessary as it is with PC and GFRC panels.
5. Kicker as needed if spandrel depth is large.
6. Hanger in base structure eliminates vertical component of kicker force in panel backup frame. Alternatively, coordinate design between SER and SSE.

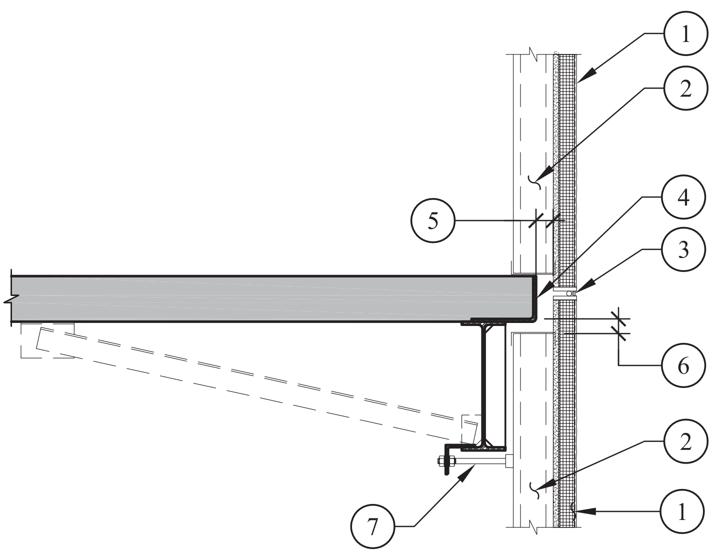
Fig. 11-2. EIFS spandrel panel attachment strategy.



NOTES:

1. EIFS full-story panel.
2. Backup frame, integral with panel.
3. Structure in backup frame to carry frame loads to backup attachments.
4. Joint between story panels must accommodate vertical movement of spandrel beam and lateral movement from inter-story drift of frame.
5. Eccentric bearing attachment. In-out adjustment provided by shim and overlap with spandrel flange. Vertical adjustment with leveling bolt and steel shims.
6. Tie-back connection at tops of panels. Vertical slots to provide for deflection of spandrel beam and horizontal slots provide for inter-story drift.

Fig. 11-3. EIFS full-story panel attachment strategy.



NOTES:

1. EIFS full-story panel.
2. Backup frame, integral with panel.
3. Joint between story panels must accommodate vertical movement of spandrel beam and lateral movement from inter-story drift of frame.
4. Pour stop. Bent plate or light-gage metal. Needs in-out adjustment.
5. Clearance needed between slab edge and back of EIFS sheathing.
6. Clearance needed between underside of slab and top of backup frame.
7. Tie-back connection.

Fig. 11-4. EIFS full-story panel "in-fill" strategy.

11.3.1 Architectural Decisions

Architectural decisions can dictate the strategy for supporting EIFS panel frames. Considerations include aesthetics, location of joints, joint width, panel configuration, drainage methods, story height, erection procedures, and economics.

The design of wall panel layout includes a balance of structural support methods, panel dimensions, and joint spacing. The use of larger panels can reduce the number of joints, but the building and façade movements and tolerances must, therefore, be accounted for in fewer locations, and joint widths tend to be wide. If thinner joints are desired, smaller and more numerous panels are usually necessary. A balance between the number of connections required for larger quantities of small panels and the more difficult installation of smaller quantities of larger panels must be considered. In either case, panel layouts that are repetitive in finish, shape, and size will often prove more economical.

Consideration must be given to the erection procedure for EIFS panels. Stresses induced from lifting and handling during panel manufacturing and installation can often control the design of the panel, and appropriate planning and coordination is therefore required.

As discussed in the "General Description" section, deficiencies have been exhibited in the performance of EIFS with regard to moisture penetration. Panel systems should be designed with appropriate consideration of drainage mechanisms.

11.3.2 Movement Requirements

Both façade and building frame movements must be accounted for in the design of EIFS panel connections. Movement can be from a combination of the effects of temperature, moisture, vertical and lateral loads, and deflection of the building frame members.

A high coefficient of thermal expansion makes EIFS prone to volume changes. Joint widths should be determined with respect to panel sizes, joint spacing, and the movement of both the panels and the building frame. Common practice includes a minimum joint width of four times the anticipated movement, but not less than ¾ in. Without proper consideration of movement and tolerances, joints can vary greatly in width and the sealants tend to crack.

Limiting out-of-plane panel movements from deflections of the supporting steel frame members to less than $L/240$ is usually sufficient for most EIFS panels. There is no industry rule of thumb for allowable vertical movement due to support deflections of the primary building structure, but the structural engineer of record (SER) should consider the deflection and joint criteria specific to their particular system. Floor slab deflection should also be taken into account, especially with large slab overhangs. Common methods of providing allowance for vertical movement of the floor slab are panel joints and vertical slots in the attachments.

11.3.3 Field Adjustability for Tolerances and Clearances

Attachments for EIFS panels require field adjustments to account for construction variations including the fabrication and erection tolerances of the steel frame. The magnitude of the necessary adjustments depends on the number of stories and the span lengths as discussed in Chapter 4. Suggested adjustments include:

- Means to adjust the slab edge closure plate in or out relative to the spandrel beam.
- Means to adjust the location of the EIFS panel frames in or out relative to the slab edge.
- Means to adjust the location of attachment anchors both vertically and horizontally.

Attachments to the building frame should be adjustable in three directions. Methods of vertical and horizontal adjustment for EIFS panels are very similar to those of aluminum curtain wall systems detailed in Chapter 9.

11.3.4 Durability

The performance of EIFS to date suggests that designers should consider the possibility of attachments being exposed to moisture, and protect them with hot-dip galvanizing or a zinc-rich coating system.

11.3.5 Fire-Safing

As discussed previously in this Design Guide, fire-safing is sometimes required to fill the gap between the floor slab edge and façade systems. Noncombustible fibrous material is used to pack the gap and a fire and smoke sealant is applied on the top surface of the insulation.

A larger gap between the slab edge and the back of the back-up frame to accommodate construction tolerances leads to the potential of having a wide gap if as-built conditions move the back-up frame away from the slab. Designers should balance the cost of adjustable slab edge items with the cost of fire-safing.

11.4 DESIGN RESPONSIBILITIES FOR EIFS PANELS

As discussed in Chapter 3, it is important that the design team understands the design responsibilities for the support of EIFS panels. Contracts can define the design responsibilities for each project, and one approach commonly found on projects with EIFS is discussed in the following text.

Architect—The architect normally has responsibility for the following:

- The EIFS panel finish, shape, and size.

- The performance of the EIFS panel system with respect to moisture and thermal protection of the building.
- The selection of the EIFS type (panels or site-applied).
- The location of horizontal soft joints (thus influencing the system of support framing).
- The dimension of the horizontal soft joints and performance of sealant joints (in consultation with the SER to understand practical limits for controlling movement from the steel frame and support assembly).
- The EIFS panel plan location with respect to column and spandrel center lines.
- The acceptable tolerances for erected and adjusted location of EIFS panels.
- The test method for performance specifications of the panel system.

Structural Engineer of Record (SER)—The SER normally has responsibility for the following:

- The design of the primary building structure, including the slab, slab edge detail, spandrel beam, roll beams, kickers, hangers, etc., for the forces imposed by the EIFS panel system with due consideration to stiffness requirements.
- Providing field adjustable items at the slab edge accommodating some, if not all, of the frame erection tolerance. (The amount of adjustability should be coordinated with the requirements in the EIFS specification, which is especially important if the panel back-up frame is in-filled between floor slabs.)
- Indicating on the structural drawings pertinent assumptions and limitations about the loads from the panels.
- Indicating load support points on the contract drawings.
- Providing the architect with the vertical and in-plane movements that the EIFS panel frame connections must accommodate for structural deformations (for inclusion in the EIFS panel specifications and/or design).
- Providing the architect with expected structural design deformations at joints.
- Review and approval of shop drawings and field erection drawings for the effect of the EIFS panels and attachments on the primary building structure.

General Contractor—The general contractor normally has responsibility for the following:

- Setting the benchmarks from which the EIFS panel installer refers when aligning EIFS work.
- EIFS panel erection, in many cases.

EIFS Panel Manufacturer and SSE—The EIFS panel manufacturer and specialty structural engineer (SSE) normally have responsibility for the following:

- The design of the EIFS components, the substrate, the attachments of the EIFS to the substrate, and the attachments that connect the substrate to the primary building structure.
- Producing shop drawings, which typically include but are not limited to the following:
  - Dimensions and locations of panels.
  - Panel frame type, including metal stud (or structural steel) sizes, fastening methods, and welding locations.
  - The EIFS specifications, including finish color, rigid expanded polystyrene insulation board (EPS) type and thickness, and type of base coat.
  - Erection procedure.
- EIFS panel erection, in some cases.

Chapter 12
Thin Stone Veneer Façade Systems

12.1 GENERAL DESCRIPTION OF THIN STONE VENEER FAÇADE SYSTEMS

The advancement of stone cutting systems in Italy in the 1980s led to thin stone veneer façade systems, which became popular in the United States (Benovengo, 1988). Where previously the thickness norm for stone veneer was 4 to 6 in., the new cutting systems made thicknesses of 3 cm common, with even thinner slabs being bonded to back-up materials to make composite stone cladding units. Additionally, the cutting systems have the capability to fabricate thin units with very large slab dimensions, and these lighter stone units have enabled a variety of ways to assemble and install the units into a façade system.

Whereas the thicker 4 to 6 in. stone units can be installed as a typical masonry cavity wall as described in Chapter 7, the thin stone veneer installations usually employ one of three systems.

Truss-supported stone panel systems—These systems have evolved significantly since their introduction. The first systems used steel trusses fabricated with relatively light members. These trusses often spanned from column to column, and the outside face was clad with thin stone units to form the spandrel panel. Many of the first installations relied solely on the stone and the sealant at the unit joints to form a barrier wall without the benefit of a back-up water or air barrier system. The truss and stone were usually prefabricated into panels, allowing them to be erected with the efficiency of precast concrete panels, but with panels that were significantly lighter and many options in stone color and texture.

Prefabricated panels allow fabrication of the wall system to start in the shop earlier in the construction schedule and arrive at the site in time for faster enclosure of the building. The trend toward prefabrication has led to designers integrating other essential components of the exterior wall system into the panel, including insulation, air and moisture barriers, drainage planes, and flashings. The configuration of the truss must consider how these other wall components will be integrated into the panel, and how the panel-to-panel joints can be sealed.

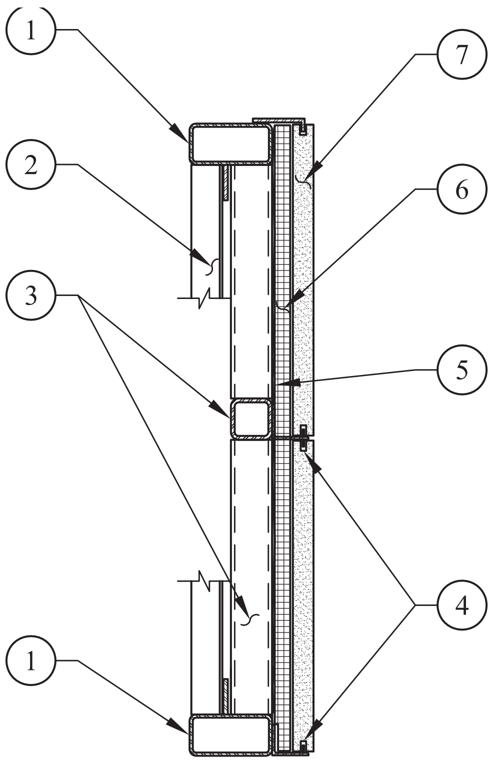
The supporting frames for the prefabricated panels are not always steel trusses. Rigid steel frames may form the framework for story-tall panels, or for column cover-type panels. Panels are also made from cold-formed light-gage steel stud framing. Steel angles, channels, or hollow structural sections may be integrated into the cold-formed stud framing as necessary to accept stone support clips and to strengthen the panel. Thin steel straps may form diagonals on the face of the studs for added rigidity. Sometimes a thin steel sheet on the outside face of the studs provides rigidity, and serves as the air and vapor barrier. Alternatively, exterior gypsum sheathing can be used in lieu of the steel sheet. Figures 12-1 and 12-2 illustrate typical panel sections.

Rail-supported stone units—Another manner in which thin stone veneers are incorporated into a wall system is by hand setting each stone in the field on a field-installed rail system supported from a back-up wall. The rails, usually steel or aluminum, are located in the cavity of the wall system behind the stone and in front of the air and moisture barriers. The rails are kept relatively thin to keep the cavity from being too large. Thus, the rails are usually supported from the back-up wall at 2- to 3-ft intervals. The rails may be oriented vertically, horizontally, or both in a given system.

Each stone unit is individually supported, usually with two gravity clips supporting the bottom of the stone unit and two lateral clips supporting the top. Sometimes the joints between units are left open and the cavity compartmentalized to form a pressurized rain screen system.

Stone units supported in aluminum curtain wall components—A third way to use thin stone units in a façade wall system is to use aluminum extrusions to form mullions and support rails in a system much like an aluminum curtain wall. The stone units are individually supported similar to the rail system previously described. This system differs in that the aluminum extrusions are sized such that they can span between floors for out-of-plane forces. This system also integrates a drainage plane or cavity, air and water barriers, and flashings behind the stone.

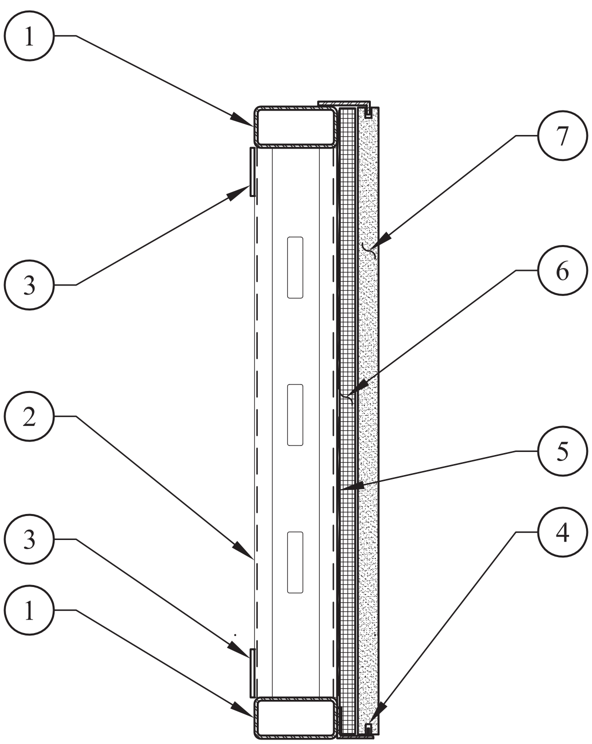
All three systems described here rely on a system of stone anchors to attach the individual stone units to the supporting panel, rails, or mullions. Anchors are either stainless steel or aluminum. Often the stone units are fabricated with kerfs in selected sides to allow support clips to engage the stone and resist out-of-plane forces. The thinness of the edges makes for tight tolerances with respect to fabrication and installation of the stone support clips. Prefabrication of panels facilitates achieving these tight tolerances. For field-installed stone units, such as with the rail system and some curtain wall systems, the stone attachments to support frames and support frame attachments to primary structure must have ample adjustability.



NOTES:

1. Truss panel chord.
2. Truss diagonals.
3. Steel grid for stone support.
4. Stone anchors.
5. Metal sheet membrane.
6. Insulation.
7. Stone units.

Fig. 12-1. Example of thin stone veneer panels supported on steel truss framing.



NOTES:

1. Truss panel chord.
2. Cold-formed steel stud panel.
3. Steel strap diagonals.
4. Stone support anchors.
5. Metal sheet membrane.
6. Insulation.
7. Stone unit.

Fig. 12-2. Example of thin stone veneer panels supported on steel stud framing.

12.2 STRATEGIES FOR SUPPORT OF THIN STONE VENEER SYSTEMS

The strategy to support thin stone veneer systems on steel-framed buildings depends on which one of the three system types the project uses. Each uses a strategy similar to other façade systems described in earlier chapters.

Prefabricated panel frames, whether made from steel trusses or cold-formed steel studs, are typically attached to the primary building structure through the use of fixed and movable connections with concepts similar to the attachment of other panel systems such as precast concrete wall panels or GFRC panels. As with other façade systems, it is not recommended to use more than two bearing connections per panel, as the reactions would become statically indeterminate. Figures 12-3 and 12-4 illustrate common methods of panel attachment.

Figures 12-5 and 12-6 demonstrate a typical method of a truss spandrel panel support. Threaded studs and leveling nuts are often used at the gravity support locations. Bearing is provided by hanging the panel from brackets welded to the primary building columns. Lateral support is provided by "push-pull" connections at the bottom corners and along the truss at intervals, if necessary. Slotted holes and the flexibility of the push-pull rods prevent unwanted restraint in the plane of the panel. Slotted holes or other provisions should allow horizontal movement in the plane of the panel at one of the bearing points to prevent restraint from thermal strains, especially in long trusses.

Figure 12-7 illustrates a common method of rail system attachment. Rails supporting thin stone units are attached to the primary structure via a back-up wall framing system. For example, a reinforced masonry back-up can be supported and anchored to the primary structure as described for back-up walls in a masonry cavity wall system. Each story of the back-up wall is supported by, and anchored to, the floor framing (slab) and laterally braced by the floor framing above in such a way as to allow both vertical and horizontal in-plane movement between the top of wall and the floor framing above. This accommodates the in-plane interstory drift. The continuous face of the masonry back-up wall allows for application of an air/water barrier and fastening the rail anchors at any spacing.

Alternatively, steel studs can be used to form the back-up with proper attention to the coordination of the stud spacing and rail anchor spacing. Additionally, the fastener design for rail anchors to metal studs must account for the gypsum sheathing that is usually part of a stud wall back-up system. Heavy-gage studs or steel posts may need to be integrated strategically into the back-up wall system.

Thin stone veneer systems that employ aluminum curtain wall type components are attached to the primary frame as described in Chapter 9.

12.3 PARAMETERS AFFECTING THE DESIGN OF THIN STONE VENEER PANEL SUPPORTS

Parameters affecting the design of thin stone veneer panel supports include:

- Architectural decisions.
- Erection procedures.
- Movement requirements.
- Field adjustability for tolerances and clearances.
- Durability.
- Fire-safing.

12.3.1 Architectural Decisions

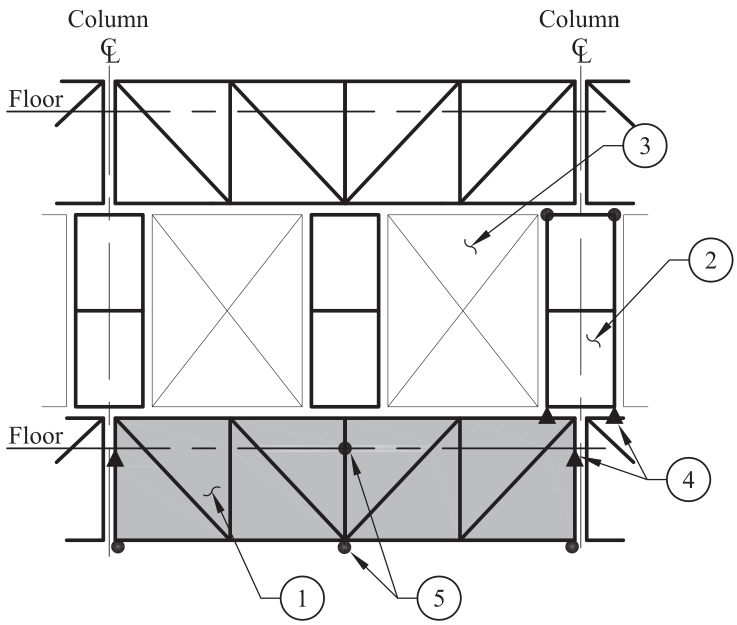
Architectural decisions can dictate the strategy for supporting thin stone veneer panel frames. Considerations include aesthetics, location of joints, joint width, panel configuration, stone unit size, drainage methods, and story height.

12.3.2 Erection Procedures

Consideration must be given to the erection procedure as this may dictate which of the support strategies is best for the project. Primary to this is whether a panelized system of erection is desirable and achievable. Panelized systems allow faster erection and, perhaps, better quality of stone work. However, incorporating drainage planes, air/water barriers, and flashings—and, as with all prefabricated wall panel systems, achieving continuity of the air/water barriers, and flashings—can be difficult. Thin stone veneer panels set on rails and stick-built curtain wall assemblies make it easier to achieve continuity of the air/water barriers and flashings but require access from the outside, usually from scaffolding. The size and location of the project will affect which system is selected for the project. Designers should be aware that past projects have changed strategies during the bidding process.

12.3.3 Movement Requirements

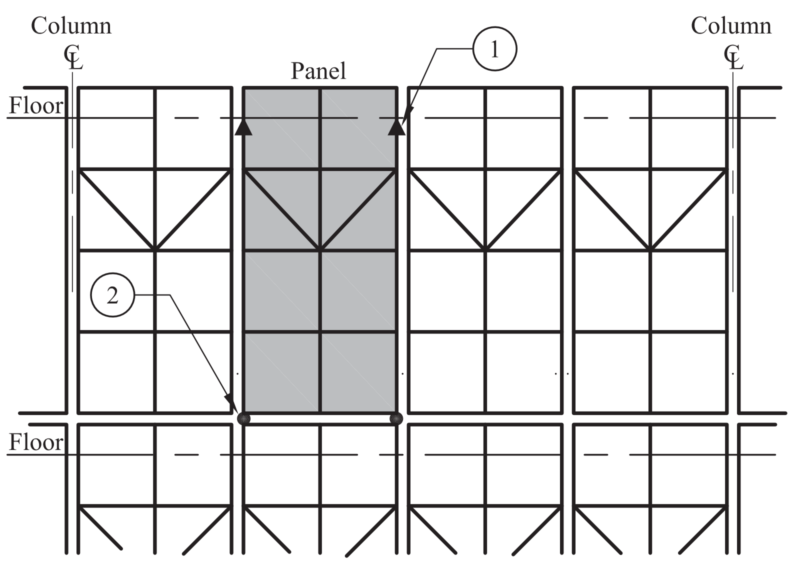
Both façade and building frame movements must be accounted for in the design of panel connections. Movement can result from a combination of the effects, including temperature, moisture, vertical and lateral loads, and deflection of the building frame members. Strategies for accommodating relative movements between panels and the primary frame are similar to those recommended for other panel-type façade systems in Chapters 8 and 10.



NOTES:

1. Truss spandrel panel.
2. Strong-back panel.
3. Window openings.
4. ▲Gravity support points.
5. ●Tie-backs (out-of-plane restraint only).

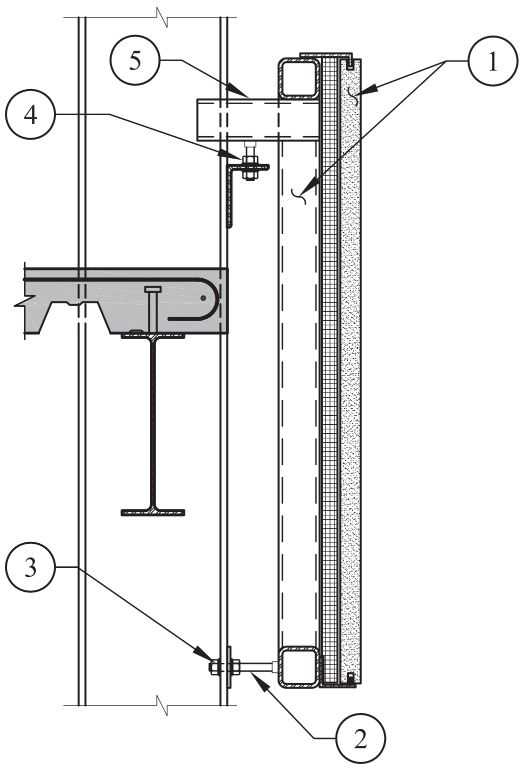
Fig. 12-3. Thin stone veneer panel support concepts—spandrel panels.



NOTES:

1. ▲ Gravity support points.
2. ● Tie-backs (out-of-plane restraint only).

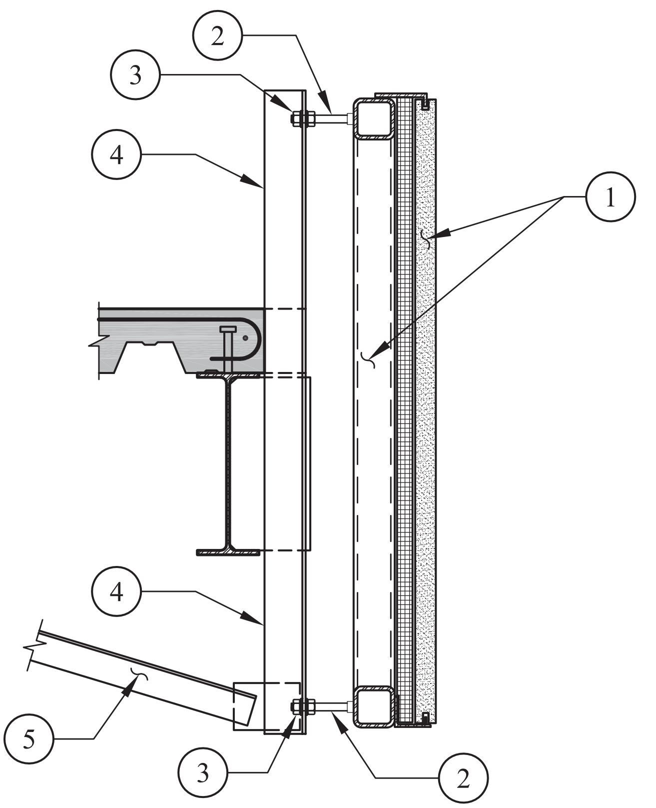
Fig. 12-4. Thin stone veneer panel support concepts—wall panels.



NOTES:

1. Truss panel.
2. Lateral tie-back for out-of-plane support.
3. Vertical slotted holes in column flange.
4. Slotted hole in angle leg for in-out adjustment.
5. Gravity load bracket.

Fig. 12-5. Example of truss spandrel panel support concept at column.



NOTES:

1. Truss panel.
2. Lateral tie-back for out-of-plane support of panel.
3. Vertical slotted holes in strong-back.
4. Strong-back angle to receive out-of-plane ties.
5. Kicker at strong-back.

Fig. 12-6. Example of truss spandrel panel support concept at mid-span.

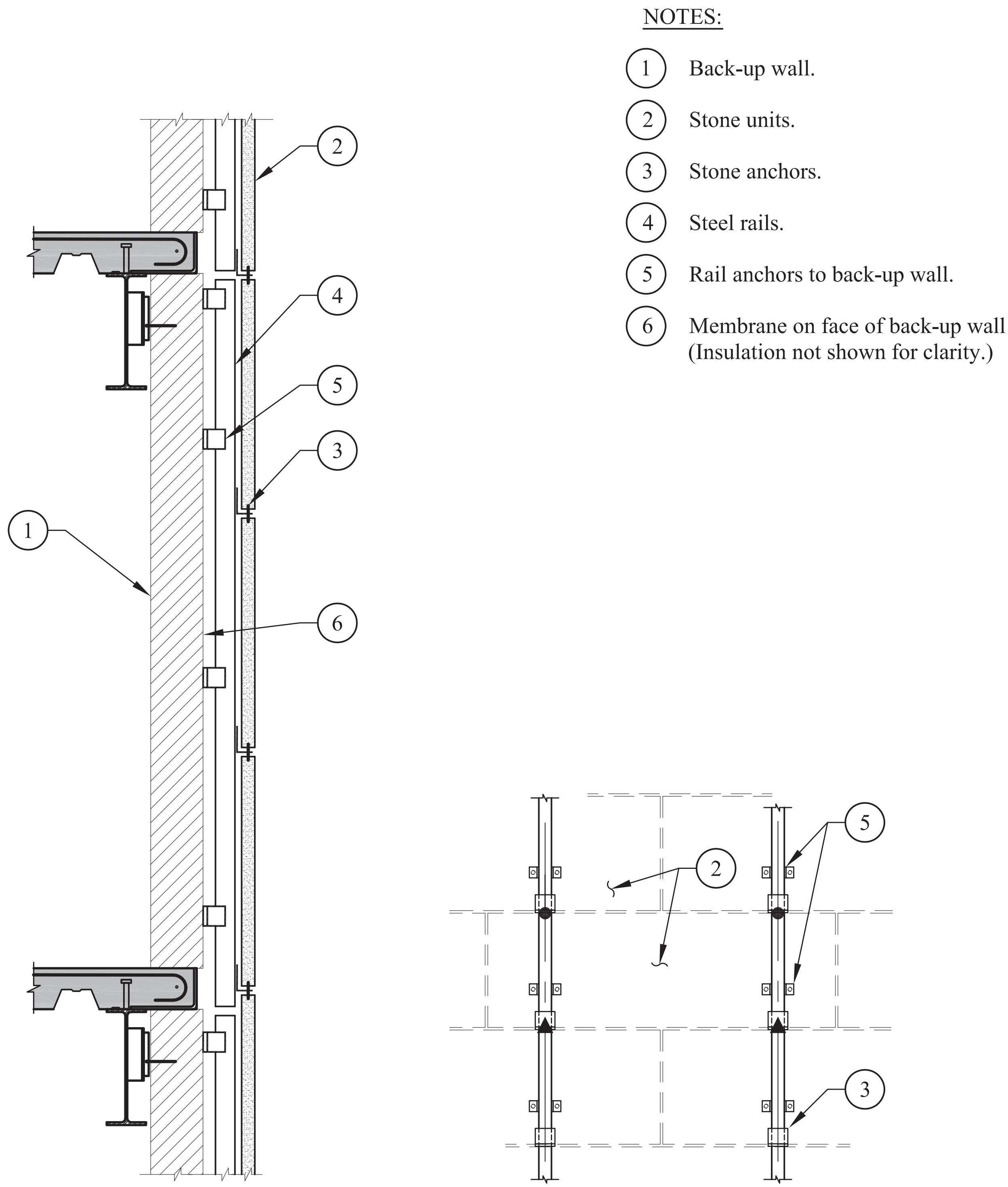


Fig. 12-7. Example of thin stone veneer support on rails.

For projects with both spandrel panels and column cover-type panels, the joints must accommodate the relative movement between panels. The magnitude and the type of joint movement (opening, closing, and translation) will depend on how the panels are attached. For example, if the column cover-type panel rotates with the column, the joints will open and close at the corners of the panel. If the panel is anchored at just the top or bottom allowing translation relative to the column at the other end, the joints will have horizontal translation.

12.3.4 Field Adjustability for Tolerances and Clearances

Attachments for thin stone veneer panels require field adjustability to account for construction variations, including the fabrication and erection tolerances of the steel frame. The magnitude of the necessary adjustments depends on the number of stories and the span lengths as discussed in Chapter 4. Suggested adjustments include:

- Means to adjust the slab edge closure plate in or out relative to the spandrel beam.
- Means to adjust the location of the panel frames in or out relative to the slab edge.
- Means to adjust the location of attachment anchors both vertically and horizontally.

Attachments to the building frame should be adjustable in three directions. Methods of vertical and horizontal adjustment for thin stone veneer panels are very similar to those of GFRC and other lightweight panel systems as detailed in Chapter 10. Projects have been completed successfully with a minimum of 2 in. of horizontal adjustability and 1 in. of vertical adjustability in the attachments (Gulyas, 1988). However, project-specific conditions may affect the appropriateness of these values for other projects.

12.3.5 Durability

The performance of thin stone veneer systems relies heavily on the performance of the anchors of the stone units to the support frames, rails, or mullions. The anchors will be exposed to moisture and thus are usually made of stainless steel and sometimes aluminum.

The anchorage of panels to the primary frame may be on the exterior side of the moisture barrier in the system or, if inside it, very near a joint in the barrier as they are often located near the panel joints. Portions of these attachments that will be exposed to moisture and hidden (or difficult to expose for inspection) should be corrosion resistant.

12.3.6 Fire-Safing

Fire-safing must be addressed for thin stone veneer systems made with panels or with curtain wall type framing, both of which typically run by the edge of the slab. As discussed previously in this Guide, fire-safing is sometimes required to fill the gap between the floor slab edge and façade systems. Noncombustible fibrous material is used to pack the gap and a fire and smoke sealant is applied on the top surface of the insulation.

A larger gap between the slab edge and the back of the backup frame to accommodate construction tolerances leads to the potential of having a wide gap if as-built conditions move the back-up frame away from the slab. Designers should balance the cost of adjustable slab edge items with the cost of fire-safing.

12.4 DESIGN RESPONSIBILITIES FOR THIN STONE VENEER FAÇADE SYSTEMS

As with other façade systems, it is important that the design team understands the design responsibilities for the support of thin stone veneer faced elements. Contracts can define the design responsibilities for each project, and one approach commonly found on projects with thin stone veneer façades is discussed in the following text.

Architect—The architect normally has responsibility for the following:

- Selection of the stone unit finish, shape, and size.
- Selection of the panel, rails, or mullion layout.
- The performance of the wall system with respect to moisture and thermal protection of the building.
- The selection of the dimension of joints and performance of sealant joints (in consultation with the SER to understand practical limits for controlling movement from the steel frame and support assembly).
- The plan location of wall system components with respect to column and spandrel centerlines.
- The setting of acceptable tolerances for erected and adjusted location of wall panels.

Structural Engineer of Record (SER)—The SER normally has responsibility for the following:

- The design of the primary building structure, including the slab, slab edge detail, spandrel beam, roll beams, kickers, hangers, etc., for the load imposed by the thin stone veneer wall system with due consideration to stiffness requirements.

- Providing the architect with expected structural design deformations at joints.

- Providing the architect with the vertical and in-plane movements that the attachment connections must accommodate for structural deformations.

- Providing field adjustable items at the slab edge accommodating some, if not all, of the frame erection tolerance. (The amount of adjustability should be coordinated with the requirements in the wall assembly specification.)

- Indicating on the structural drawings pertinent assumptions and limitations about the loads from the wall system.

- Indicating load support points on the contract drawings.

- Review of shop drawings and field erection drawings for the effect of the wall system and attachments on the primary building structure.

General Contractor—The general contractor normally has responsibility for the following:

- The locating of benchmarks and offset lines from which the wall installer refers when aligning his work.

- The coordination of the design and construction of the back-up wall and the rail system, if each is provided by a different subcontractor.

Wall System Manufacturer/SSE—For panel systems and curtain-wall-type systems, all aspects of the stone/wall panel assembly should be a sole-source responsibility of a subcontractor. For rail-type systems, a sole source responsibility is recommended at least from the face of the back-up wall out. The back-up wall may be included too, or it may be another subcontractor.

One or more SSEs, usually working for the wall contractor, is responsible for the design of the wall structural components, attachments of the stone units to the frames, and the attachments that connect the frames to the primary building structure. An SSE is responsible for the back-up wall design for rail-type systems.

APPENDIX A
Results of Finite Element Models to Study the Effect of Slab/Deck Translational Restraint on Spandrel Beams

A.1 GENERAL DESCRIPTION OF MODELS

This appendix presents the results of finite element analyses used to establish guidelines for the analysis of torsionally loaded wide-flange beams with a top-flange diaphragm restraint. Current design guidelines address the analysis of bare steel beams loaded in torsion but do not address the restraining effect of either a roof deck or composite slab. To provide the designer with useful design guidelines for such situations, the author analyzed 24 finite element models. The finite element models used thin shell elements to model six wide-flange sections of 10-ft and 30-ft spans for the effects of concentrated and uniform torsional loading.

The finite element models assumed a pin (three translation degrees of freedom restrained, no rotation or warping restraint) at each edge mesh point along the web, and planer pin restraints (only vertical deflection permitted, translation in the plane of the slab/deck is restrained, no rotational restraint) along the centerline of the top flange. Figure A-1 shows an isometric of a model and schematic supports.

Composite slabs will provide rotational restraint as well as translational restraint. However, the rotational restraint is neglected in this study. This is conservative with respect to estimating rotation of the spandrel.

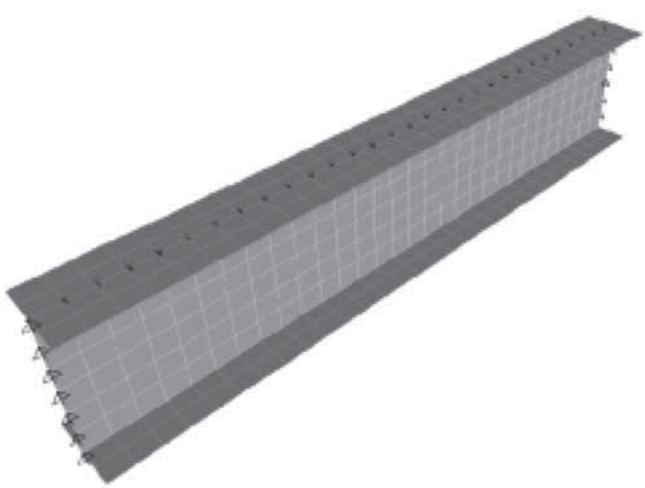


Fig. A-1. Isometric view of finite element model.

A.2 ALTERNATIVE METHODS TO APPROXIMATE TORSIONAL DEFLECTIONS OF TOP-FLANGE-RESTRAINED BEAMS

In addition to presenting the finite element results this appendix compares them to two alternative design methods. The first method is called the Modified Flexural Analogy Method. This method uses the flexural analogy method presented by Salmon and Johnson (1996) for the analysis of steel beams in torsion. However, this method accounts for the fact that the top flange cannot translate and thus the beam section rotates about its top flange instead of its center.

The Modified Flexural Analogy Method accounts for this by assuming no lateral translation of the top flange. The rotation is predicted by calculation of the horizontal deflection of the bottom half of the section and dividing this deflection by the beam depth.

The second method is called the Modified Design Guide Method. This method uses the analysis method presented in AISC Design Guide No. 9 (Seaburg and Carter, 1997), which considers the beam as a bare steel beam. It is conservatively assumed that the restrained beam rotates the same amount as from AISC Design Guide No. 9, but the rotation is assumed to occur about the top flange.

A.3 DISCUSSION OF RESULTS

Tables A-1 through A-4 present beam rotations for 10-ft and 30-ft models subjected to concentrated and uniform torsional loads. These tables also present ratios of the calculated rotations for each of the three methods.

The results show that the Modified Flexural Analogy Method provides a more accurate approximation of rotation about the top flange at relatively short spans. However, this method becomes increasingly conservative as the beam span increases. The Modified Design Guide Method better predicts rotation about the top flange for most beams as beam span increases. The only exception in the studied cases is for the W24×117, which is a deep beam with a relatively large flange width.

The ratios of predicted rotation for the methods are similar for both concentrated and uniformly applied loads.

A.4 CONCLUSIONS AND DESIGN RECOMMENDATIONS

The Modified Flexural Analogy Method requires less computation than the Modified Design Guide Method, and more accurately predicts torsional rotations about the top flange in short spans. However, the rotations computed with the flexural analogy can become too conservative for narrow-flanged beams at long spans. In such cases, the Modified Design Guide Method better predicts the torsional rotations about the top flange.

In many cases, the presence of roll beams or kickers will cause the effective torsional span length to be closer to 10 ft than 30 ft. In these cases, the Modified Flexural Analogy Method will not only require less computational effort, but will also provide a more accurate approximation of the beam's rotation about the top flange. For longer spans where this method is too conservative, the Modified Design Guide Method may provide more accurate approximations of beam rotation about the top flange.

Table A-1. Top Flange Rotation for 10-ft Span with Distributed Twist

Beam	Rotation (rad.)			Ratio		
	Finite Element Model (FEM)	Modified Design Guide Method (MDGM)	Modified Flexural Analogy Method (MFAM)	MDGM/ FEM	MFAM/ FEM	MDGM/ MFAM
W14×30	0.417	0.807	0.477	1.93	1.14	1.69
W14×43	0.178	0.349	0.209	1.97	1.18	1.67
W18×40	0.242	0.468	0.291	1.93	1.20	1.61
W18×60	0.0879	0.175	0.107	1.99	1.22	1.63
W24×62	0.0791	0.158	0.0923	2.00	1.17	1.71
W24×117	0.00865	0.020	0.0101	2.31	1.17	1.97

Table A-2. Top Flange Rotation for 10-ft Span with Concentrated Mid-Span Twist

Beam	Rotation (rad.)			Ratio		
	Finite Element Model (FEM)	Modified Design Guide Method (MDGM)	Modified Flexural Analogy Method (MFAM)	MDGM/ FEM	MFAM/ FEM	MDGM/ MFAM
W14×30	0.333	0.648	0.382	1.95	1.15	1.70
W14×43	0.141	0.281	0.168	1.99	1.19	1.68
W18×40	0.193	0.377	0.233	1.96	1.21	1.62
W18×60	0.0699	0.140	0.0858	2.01	1.23	1.64
W24×62	0.0629	0.127	0.0738	2.02	1.17	1.72
W24×117	0.00684	0.016	0.00811	2.34	1.19	1.97

Table A-3. Top Flange Rotation for 30-ft Span with Distributed Twist						
Beam	Rotation (rad.)			Ratio		
	Finite Element Model (FEM)	Modified Design Guide Method (MDGM)	Modified Flexural Analogy Method (MFAM)	MDGM/ FEM	MFAM/ FEM	MDGM/ MFAM
W14×30	20.1	25.4	42.4	1.26	2.11	0.599
W14×43	8.37	9.83	17.2	1.17	2.06	0.571
W18×40	10.7	12.9	24.3	1.21	2.27	0.531
W18×60	3.90	4.81	8.72	1.23	2.24	0.551
W24×62	4.11	5.27	7.50	1.28	1.82	0.703
W24×117	0.603	0.961	0.821	1.59	1.36	1.17

Table A-4. Top Flange Rotation for 30-ft Span with Concentrated Mid-Span Twist						
Beam	Rotation (rad.)			Ratio		
	Finite Element Model (FEM)	Modified Design Guide Method (MDGM)	Modified Flexural Analogy Method (MFAM)	MDGM/ FEM	MFAM/ FEM	MDGM/ MFAM
W14×30	5.43	6.98	10.4	1.29	1.91	0.674
W14×43	2.25	2.71	4.53	1.21	2.01	0.599
W18×40	2.88	3.56	6.29	1.24	2.18	0.566
W18×60	1.05	1.33	2.32	1.27	2.21	0.574
W24×62	1.11	1.44	1.99	1.31	1.80	0.724
W24×117	0.161	0.259	0.219	1.61	1.36	1.18

REFERENCES

American Architectural Manufacturers Association, 1989a, *Installation of Aluminum Curtain Walls*, CWG-1-89, AAMA, Schaumburg, IL.

American Architectural Manufacturers Association, 1989b, *Metal Curtain Wall Manual*, MCWM-1-89, AAMA, Schaumburg, IL.

American Architectural Manufacturers Association, 2005, *Curtain Wall Design Guide Manual*, CW-DG-1-96, AAMA, Schaumburg, IL.

ACI International, 2004, *Building Code Requirements for Structural Concrete (ACI 318-05) and Commentary (ACI-318R-05)*, ACI, Farmington Hills, MI.

ACI International (et al.), 2005, *Building Code Requirements for Masonry Structures (ACI 530-05/ASCE 5-05/TMS 402-5)*, ACI, Farmington Hills, MI.

American Institute of Steel Construction, 2005a, *Code of Standard Practice for Steel Buildings and Bridges (AISC 303-05)*, AISC, Chicago, IL.

American Institute of Steel Construction, 2005b, *Specification for Structural Steel Buildings (ANSI/AISC 360-05)*, AISC, Chicago, IL.

American Institute of Steel Construction, 2005c, *Steel Construction Manual*, 13th Ed., AISC, Chicago, IL.

American Society of Civil Engineers, 2002, *Design Loads on Structures During Construction,* SEI/ASCE 37-02, ASCE, Reston, VA.

American Society of Civil Engineers, 2005, *Minimum Design Loads for Buildings and Other Structures*, SEI/ASCE 7-05, ASCE, Reston, VA.

Arnold, C., 2006, "Seismic Safety of the Building Envelope", *Whole Building Design Guide*, available at http://www.wbdg.org/design/env_seismicsafety.php.

Ballast, D. K., 1994, *Handbook of Construction Tolerances*, McGraw-Hill, Inc., New York, NY.

Benovengo, E.A. Jr., 1988, "Design Criteria for Thin Stone/Truss Wall Systems," *New Stone Technology, Design, and Construction for Exterior Wall Systems*, ASTM STP 996, B. Donaldson, Ed., American Society for Testing and Materials, Philadelphia, PA, pp. 57–65.

Brick Industry Association, 2000, "Volume Changes-Analysis and Effects of Movement," *Technical Notes on Brick Construction,* No. 18, Reston, VA.

Brock, L., 2005, *Designing the Exterior Wall: An Architectural Guide to the Vertical Envelope*, John Wiley & Sons, Inc., Hoboken, NJ.

Construction Sciences Research Foundation, 2006, *SPEC-TEXT® Master Guide Specifications*, CSRF, Baltimore, MD.

Griffis, L.G., 1993, "Serviceability Limit States Under Wind Loads," *Engineering Journal*, 1st Qtr., AISC, Vol. 30, No.1, Chicago, IL.

Grimm, C.T. and J.A. Yura, 1989, "Shelf Angles for Masonry Veneer", *Journal of Structural Engineering*, Vol. 115, No. 3, (March), ASCE, Reston, VA.

Gulyas, S., 1988, "Truss Supported Stone Panel Systems," *New Stone Technology, Design, and Construction for Exterior Wall Systems*, ASTM STP 996, B. Donaldson, Ed., American Society for Testing and Materials, Philadelphia, PA, pp. 69–86.

International Code Council, 2006, *International Building Code*, ICC, Country Club Hills, IL.

Janakus, J., R. Huddleston, and B. Wolfe, 2006, *Steel Building—Exterior Wall Interfaces*, North American Steel Construction Conference (NASCC) Proceedings, AISC, Chicago, IL.

Lin, P.H., 1977, "Simplified Design for Torsional Loading of Rolled Steel Members," *Engineering Journal*, AISC, Vol. 14, No. 3, Chicago, IL.

Precast/Prestressed Concrete Institute, 1989, *Architectural Precast Concrete, Second Edition*, S. Freedman, A.R. Kenney, Eds., PCI, Chicago, IL.

Precast/Prestressed Concrete Institute, 1998, *Design and Typical Details of Connections for Precast and Prestressed Concrete*, A.F. Shaikh, Ed., PCI, Chicago, IL.

Precast/Prestressed Concrete Institute, 2000, *Tolerance Manual for Precast and Prestressed Concrete Construction,* MNL-135, PCI, Chicago, IL

Precast/Prestressed Concrete Institute, 2001, *Recommended Practice for Glass Fiber Reinforced Concrete Panels*, Fourth Edition, PCI, Chicago, IL.

Precast/Prestressed Concrete Institute, 2004, *PCI Design Handbook*, Sixth Edition, L.D. Martin, C.J. Perry, Eds., PCI, Chicago, IL.

Salmon, C.G. and J.E. Johnson, 1996, *Steel Structures Design and Behavior: Emphasizing Load and Resistance Factor Design*, T.M. Slaughter, Ed., Harper Collins College Publishers, New York, NY.

Seaburg, P.A. and C.J. Carter, 1997, AISC Design Guide No. 9 *Torsional Analysis of Structural Steel Members*, AISC, Chicago, IL.

Steel Deck Institute, 2000, *SDI Design Manual for Composite Decks, Form Decks, and Roof Decks,* Publication No. 30, SDI, Fox River Grove, IL.

Steel Deck Institute, 2004, *Diaphragm Design Manual,* 3rd Edition, SDI, Fox River Grove, IL.

Thomas, R.G., Jr., 1992, *Exterior Insulation and Finish System Design Handbook*, CMD Associates, Inc., Vashon Island, WA.

Tide, R.H. and N.V. Krogstad, 1993, *Economical Design of Shelf Angles,* J.H. Melander, L.R. Lauersdorf, Eds., ASTM, Philadelphia, PA.